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TESTS ON 6 ¼ INCH METAL SPARS

Air Service Information Circular, Volume VII, No. 622

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Engineering Division
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Historical study to determine the relative merits of different types of metal wing spar construction, and to develop methods to be used in the design of those types which the tests indicated to be the most satisfactory for use in practical design.

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TESTS ON 6 $\frac{1}{4}$ INCH METAL SPARS

Appendix I. Logs of Tests on 6 1/4 Inch Spars

Appendix II. Requirements for Experimental Metal Spars for Pursuit and Observation Loadings

(AIRPLANE BRANCH REPORT)

▽

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TESTS ON 6¼-INCH METAL SPARS

OBJECT AND SCOPE

1. The study discussed in this report was made to obtain data on the relative merits of different types of metal wing spar construction, and to develop methods to be used in the design of those types which the tests indicated to be the most satisfactory for use in practical design. It is believed that the best type of construction to be used for any given airplane depends upon many factors, of which the size of the airplane and the intensity of loading are among the most important. Although it is intended to cover ultimately the entire range of airplane sizes, the work covered by this report pertains only to spars of a size suitable for use in single bay externally braced observation airplanes.

2. This is the second report on the spar tests covered, the earlier report being published as Air Corps Information Circular, Vol. VI, No. 590 [1] (Serial Report 2665), "Progress Reports on Experimental Metal Spars," by A. S. Niles and E. C. Friel. That report gave the more obvious results of the tests, but was written before the test results could be subjected to more intensive study. The present report includes not only the more important results given in Air Corps Information Circular, Vol. VI, No. 590, but also gives the results of tests to determine the quality of the material in the spars tested, computations of unit stresses developed, geometrical properties of the cross sections and similar quantities, studies of the stiffness characteristics of the spars, an investigation of the characteristic types of failure, the development of methods to be used in the design of the two types which gave the most favorable test results and revised recommendations regarding future work in the study of the problem of metal construction.

CONCLUSIONS

3. The principal conclusions reached as a result of the study recorded in this report are the following:

(a) The methods of test and study used in this project are well adapted to determining the relative merits of different types of metal spar construction, though additional data should be called for with the spars and certain minor changes in the test procedure are required to obtain the maximum of information. This additional data and changes in procedure are discussed in paragraphs 156 to 173.

(b) The most promising types of metal spar yet tested are the duralumin box and the duralumin channel truss, and empirical rules given in paragraphs 110 to 143 have been developed for the design of these types. Though the best of the metal types tested, neither of these types has shown up as well as the conventional wood box, nor appreciably better than the conventional wood I beam. Two or three other types of metal construction have shown great promise but have not yet been developed to the degree of excellence attained by the types mentioned, and no rules for the design of these other types have yet been formulated.

(c) The unit stress in the extreme fiber at failure is not a measure of the merit of a type of spar construction.

(d) The unit stress at failure varies greatly with the type of construction although the material used may be the same. The study of the effect of the type of construction on the unit stress at failure, called the study of "form factors," is essential for the development of satisfactory rules for metal spar design. To date little work of this character has been done, and much must be accomplished in this line before the design of metal spars is put on as satisfactory a basis as that on which the design of wood spars rests at the present time.

(e) Owing to the importance of the stiffness of spars in computing the secondary bending moments to which they will be subjected it is necessary to obtain for each type a stiffness factor which is the ratio of the value of EI computed from its deflection to the value of EI computed from the dimensions of its cross section and the modulus of elasticity of the material used. Average values for the stiffness factors for duralumin box and channel truss types of spar have been computed and are recommended for use in design in paragraphs 116 and 139. Further study of this quantity with reference to other types of construction and its variation with the type of loading used is needed.

(f) The study of the tendency of spars to fail by lateral buckling of the compression chord has shown that failure is likely to occur whenever the compression chord is loaded to the unit stress corresponding to the failing load of a pin-ended column of length equal to the distance between points of lateral support of that chord and with a radius of gyration equal to that of the compression chord about the major axis of the spar. It has also been shown that the probability of lateral buckling decreases with increased torsional strength of the spar. This study did not cover the effect of the wing ribs in providing lateral support to the compression chord, no tests being made which would throw light on that phase of the problem.

[1] This report has a number of references to Air Corps Information Circular, Vol. VI, No. 590 (Serial Report 2665), "Progress Reports on Experimental Metal Spars" by A. S. Niles and E. C. Friel, and Air Corps Information Circular, Vol. VI, No. 598 (Serial Report 2777.) "Compressive Strength of Duralumin Channels" by Roy A. Miller, both of which have been published.

LOADING USED

4. When this series of tests was projected, it was considered that the class of spars on which information was most needed was the spar of medium depth suitable for use in single bay airplanes of about 4,500 pounds gross weight. The loading used was developed from a study of the available stress analyses of airplanes of this general character, an attempt being made to obtain a combination of shear, axial load, and primary and secondary bending moment comparable with the maxima encountered in airplanes of this class. The details of this loading are given below in the description of the tests.

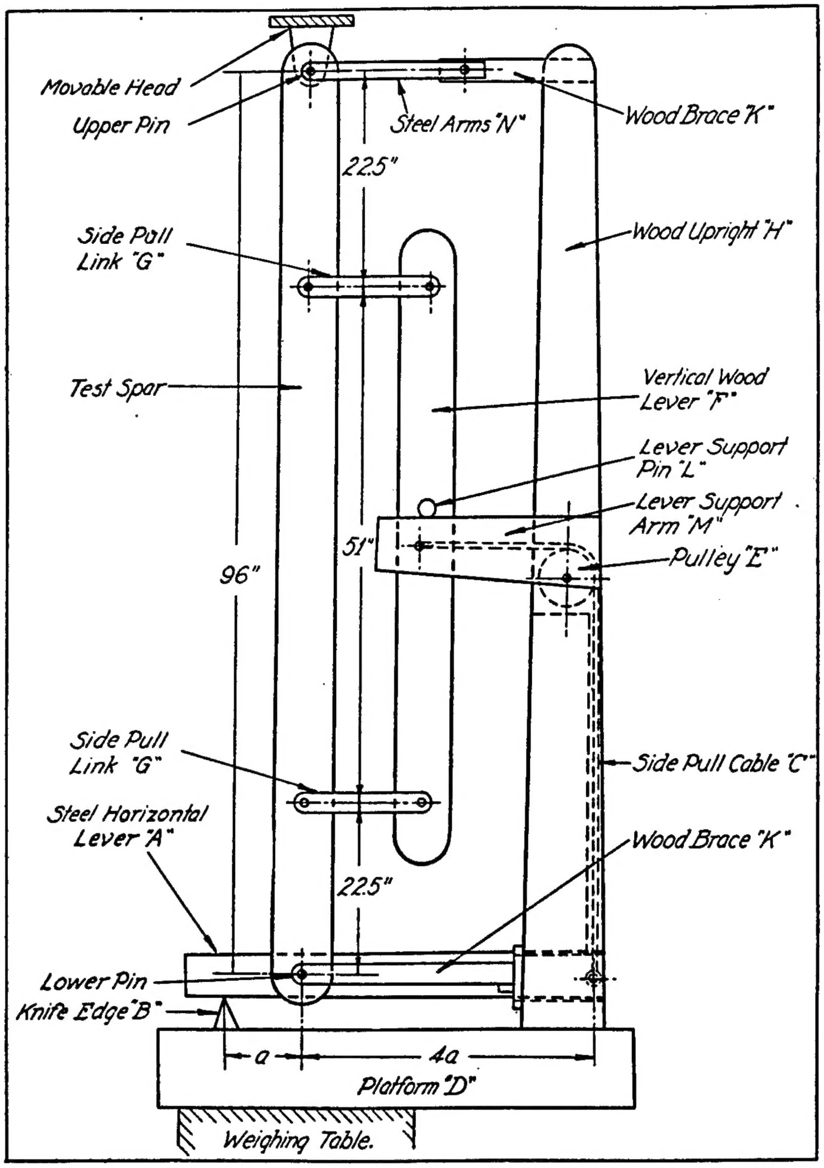

FIG. 1.—Diagram of test set-up

DESCRIPTION OF TESTS

5. The method of test is shown diagrammatically in Figure 1, while Figures 2 and 3 show the views of the test rig with a spar in place. The test rig was devised to test the specimen under a combination of axial and side load, the ratio of these two loads being fixed, but the secondary bending moments depending, as they do in practice, on the stiffness of the specimen. The axial load was applied to the specimen from the movable head of the testing machine, through a ¾-inch pin. At the lower end of the spar the axial load was taken out through a similar pin joint, but the lower pin, instead of being directly supported by the weighing table of the testing machine, rested on the horizontal steel lever A. This lever was supported, in turn, by the knife edge B and the side pull cable C. The dimensions of the steel lever A were such that the load in the cable C was always one-fifth of the axial load in the specimen. The knife edge B transmitted its load directly to the platform D resting on the weighing table of the testing machine. The cable G passed over the pulley E to the vertical wood lever F which divided

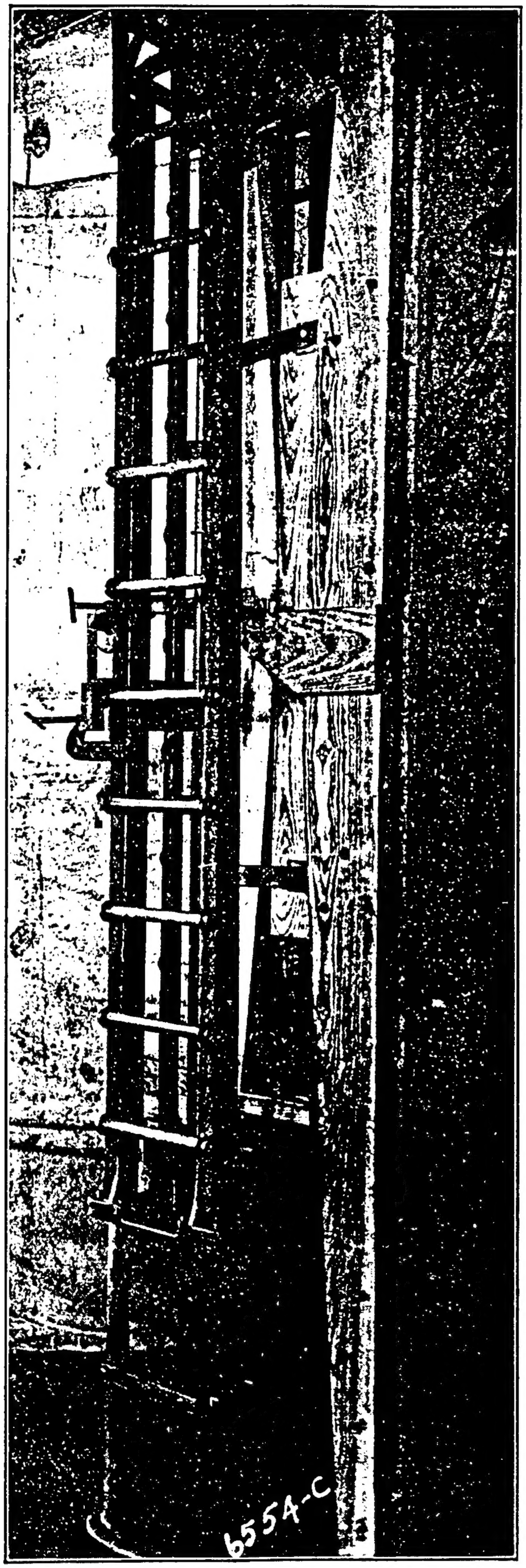

Fig. 2

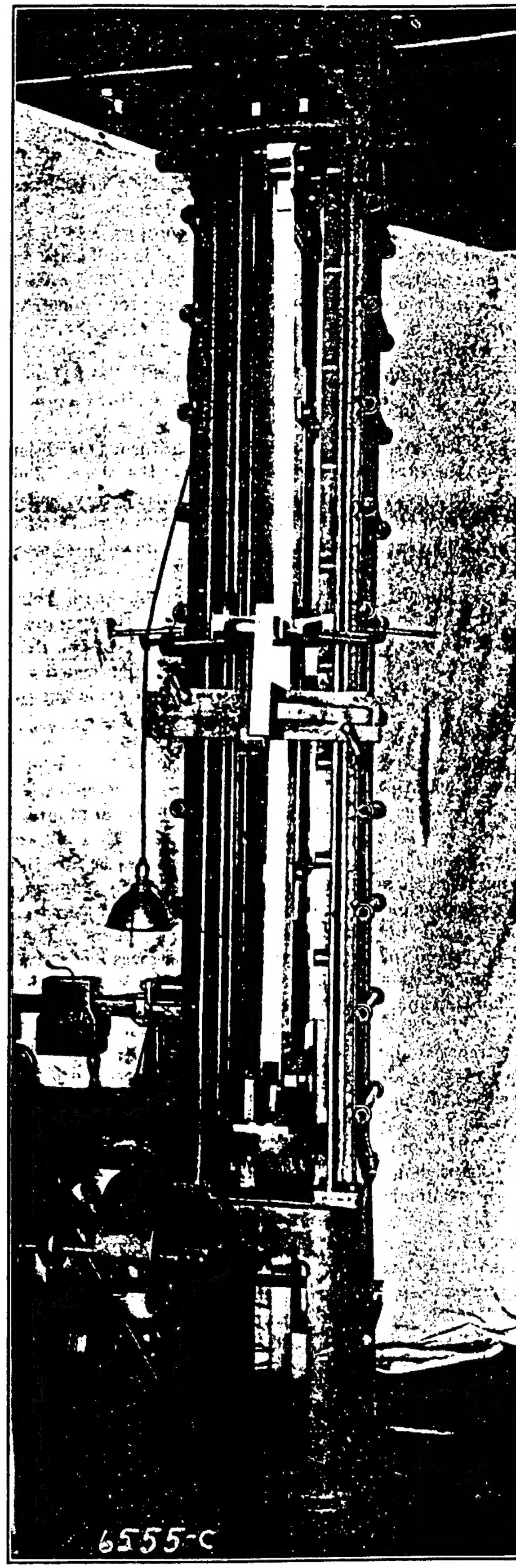

Fig. 3

the load in the cable into two equal transverse loads applied to the test specimen through the side pull links G, G. The vertical component of the load on the pulley E was carried to the platform D and thence to the weighing table by the wood upright H. The horizontal component is carried by the wood upright H acting as a beam supported by the wood braces K, K, which press against the pins at the ends of the test specimens, the upper brace and the steel arms N forming a single rigid member. As the compressions in the wood braces and the tensions in the side pull links are numerically equal, the system of horizontal forces is in equilibrium. The wood lever F is prevented from dropping by being supported by the lever support pin L resting on the lever support arms M on each side, that are rigidly connected to the wood upright H.

6. The whole arrangement is shown in Figures 2 and 3. These pictures were taken before the jig had attained its final form, so that the platform D is absent and the wood upright H rests on the floor. This earlier method of testing proved unsatisfactory, as the load from the movable head was not all carried by the weighing table, part being carried down the wood upright to the floor. Not only did this result in the necessity of the load readings on the beam being multiplied by the factor 1.25 to obtain the true axial load in the test specimen, but that part of the axial load that was carried to the weighing table through the knife-edge B was applied eccentrically, overloading the knife-edges on one side of the weighing table.

7. The spars were supported laterally by wood blocks faced by metal plates clamped to the uprights of the testing machine. Similar metal faced wood blocks were attached to the spars at the point of lateral support. The metal protected faces of these blocks were not allowed to come into contact as the friction between them would have decreased the transverse or bending load carried by the spar itself, but they were separated by long rivets acting as rollers. These blocks and rollers were located so there was a clearance of about one-sixteenth inch, and the spar had to deflect half that amount before resting against one of the blocks.

8. The specifications under which the spars were purchased stated that they would be tested with only one set of lateral supports, these to be located at the center of the 96-inch span. Several of the metal spars when tested in this manner, failed by lateral buckling of the compression chord, the chord assuming an S-shape as shown by spar 12–A in Figure 13. As it was contended by the designers of these spars and others that in actual practice the ribs, leading edge covering, etc., would tend to prevent this action, and that, therefore, the test conditions were unfair to the types of construction concerned, several of the spars were tested with a larger number of additional supports. At first, the custom was to test the first spar of a type with a single location of lateral support, the center of the span. If the first article failed by lateral buckling, the second article was tested with lateral supports at the third points but none at the center. Even with the reduced distance between lateral supports many of the spars still failed by lateral buckling, as shown by spar 14–B in Figure 13. The third spar of type 14 was tested with four points of lateral support spaced one-fifth of the 96-inch span apart.

9. Later in the investigation, the practice changed somewhat. If the preliminary tests showed such a low stiffness about the major axis of the spar section that the spar was almost certain to fail by lateral buckling of the compression chord, even the first spar of a type would be tested with two lateral support points. In a few doubtful cases the spar was tested first with a single lateral support until maximum load was reached. The spar was then quickly relieved of load and tested again with two points of support. In some cases this procedure seemed to have been gone through without injury to the spar in the first test, but in others the load carried when tested with two supports was so little larger than that obtained with a single support that it seems that the spar must have been injured in the first test.

10. The question of lateral buckling of the compression chord and the proper distance between lateral support points in tests has caused much discussion but will not be gone into here in any more detail as it is discussed more fully in Air Corps Information Circular, Vol. VI, No. 590.

11. Deflections at the center of the test specimen were measured to the nearest thousandth of an inch by a deflectometer similar to a Wissler dial. In the early stages of the test the deflections were read as previously determined values of the axial load were attained. In the later stages the weighing beam was kept in balance and the axial load read as previously determined deflections were reached. Normally the deflections were read at thousand-pound increments of axial load in the first case, and axial load at 0.05-inch increments of deflection in the second.

12. All of the beams tested were designed for the test and to carry 20,000 pounds axial load. At this stage the transverse loads would be 2,000 pounds each. The primary bending moment would be 45,000-inch pounds, while the secondary bending would depend upon the actual deflection.

TEST RESULTS

13. The logs of the tests of the metal spars under combined bending and compression are given in Appendix I. The results of these tests are summarized in Table 1 with the results of tests in simple bending to determine the stiffness moduli (EI values) of the spars. The data of Table 1 and the more important conclusions that can be drawn from it are published in Air Corps Information Circular Vol. VI, No. 590, and are republished here as they will be constantly needed in the discussions of this report. In addition to the main tests, the results of which are recorded in Table 1, certain tests were made to determine the quality of the material in the spars. Unfortunately, these tests were not complete, and the lack of necessary information of

this character has made it impossible to obtain as much data regarding methods of design as had been hoped for. In the future, if the recommendations of this report are carried out, a complete series of such tests on "minors" to determine the quality of the material used will be required before experimental spars are delivered to the division. The results of the minor parts tests that were made are recorded in Table 2.

The column headings of Table 2 have the following meanings:

Heading	*Significance*
E_c	Modulus of elasticity of material in compression chord.
E_t	Modulus of elasticity of material in tension chord.
E_w	Modulus of elasticity of material in web.
PL_c	Proportional limit of material in compression chord.
PL_t	Proportional limit of material in tension chord.
PL_w	Proportional limit of material in web.
U_c	Ultimate strength of material in compression chord.
U_t	Ultimate strength of material in tension chord.
U_w	Ultimate strength of material in web.
YP_c	Yield point of material in compression chord.
YP_t	Yield point of material in tension chord.

TABLE 1.—*Data from tests on complete spars*

Spar 1	Figure 2	Type 3	Failure 4	Number supports 5	Ultimate load 6	Weight of 7 feet 7	δ at ultimate P	δ at 10,000 8	$\frac{EI_{xx}}{1000}$ 9	$\frac{EI_{yy}}{1000}$ 10
WOOD SPARS										
1A	10	Rectangle	Horizontal shear	1	20,625	16.3	1.105	0.465	69,000	8,440
1B	10	do	Compression followed by tension	1	20,625	16.5	1.280	.455	60,200	7,290
2A	11	Box	Crushing of compression flange at center	1	24,690	15.0	1.100	.353	85,700	11,770
2B		do	do	1	21,700	13.8	1.100	.380	85,800	10,300
2C	5N	do	do	1	22,380	13.5	1.550	.382	78,330	8,790
3A	7	T-box	Crushing of compression flange at end of ears.	1	15,400	10.0	.600	.288	64,100	12,250
3B		do	Crushing of compression flange 17½ inches from end pin.	1	14,070	10.5	.430	.275	68,650	11,900
3C		do	Crushing of compression flange 16 inches from end pin.	1	18,110	10.7	.600	.270	67,500	12,250
3D		do	Horizontal shear in webs between end fittings and side load points.	1	18,680	10.64		.228	61,600	10,180
3F		do	Shear and bending near end fitting	1	20,720	10.77	.90	.240	59,200	9,680
4A	10	I-beam	Horizontal shear	1	19,450	13.7		.287	73,500	10,220
4B	51	do	do	1	19,750	13.3		.312	74,300	10,680
4C	5H	do	Crushing of compression flange at side load point.	1	21,320	13.3	1.300	.465	75,100	9,010
4D		do	Crushing of compression flange at center	1	18,650	12.6	1.200	.525	66,700	7,830
4E		do	Horizontal shear	1	20,750	14.1		.327	84,700	10,100
DURALUMIN SPARS										
10A	46, 11	Box	Tension in rivets of compression flange 10 inches from center.	1	19,375	14.5	0.685	0.292	75,200	8,180
10B	11	do	Tension in rivets of compression 10 inches inside load point.	1	20,185	14.6	.950	.280	91,200	11,970
10C	10	do	Tension in rivets of compression flange 19 inches inside load point.	1	15,900	13.8		.296	86,600	10,970
10D	10	do	Tension in rivets of compression flange 11 inches inside load point.	1	17,010	13.8	.650	.287	88,200	11,710
10E	10	do	Tension in rivets of compression flange 20 inches inside load point.	1	16,790	12.7		.332	90,600	9,310
11A	10, 11	Tube and plate	Lateral buckling	1	15,860	13.4	1.400	.540	38,004	4,875
11B	4B, 11	do	Bending in plane of loads followed by lateral buckling.	2	16,250	12.4	1.750	.537	54,600	5,340
12A	12, 13	Bulb I	Lateral buckling	1	15,875	15.9	.600	.287	89,000	4,390
12B	4B, 12	do	do	2	19,240	16.0		.317	83,450	4,500
13A	12	Bevel I	do	1	14,375	14.5	.580	.350	72,800	4,100
13B	4D, 12	do	do	2	16,425	15.0	.800	.370	76,000	3,875
14A	4F	Plate girder	do	1	15,500	13.8	.550	.264	78,300	4,755
14B	4A, 12, 13	do	do	2	18,500	13.6	.750	.283	96,000	4,755
14C	5K	do	do	4	22,100	16.2	.800	.273	106,700	5,870
14D		do	do	1	19,690	15.81	.70	.245	133,500	6,860
15A	12	Channel truss	Lateral buckling of one panel length of compression flange 13 inches inside load points.	1	19,290	13.6	1.000	.340	81,100	8,460
16A	6P, 13	do	Crushing of compression flange 22 inches inside load points.	1	18,500	12.7	.950	.312	77,500	16,920
16B	8	do	Local buckling of compression flange near center.	1	18,050	13.06	1.05	.257	83,700	18,480
16C	8	do	do	1	18,750	13.42	1.10	.278	87,100	20,700
16D	8	do	Compression failure in plane of truss in bay near center.	1	14,850	12.07	1.15	.332	79,900	13,130
16E	8	do	Gradual bending laterally	1	14,960	11.97	1.30	.342	72,300	13,620
16F-2	8	do	Compression failure in plane of truss of bay near center.	1	16,740	12.58	.65	.361	78,000	18,350
16G	8	do	Tearing of end web member away from chord.	1	20,150	13.48	1.25	.281	84,600	20,200
16H	8	do	Tearing of 3d web member away from chord.	1	19,900	14.56	1.10	.297	87,400	19,530
17A	6S, 13	Trussed web dumb-bell.	Lateral buckling in compression flange near center.	2	16,170	10.4	.950	.446	60,650	5,525
17B	6T, 13	do	do	2	15,300	10.1	1.000	.500	61,300	5,490
18A	3M, 11	Hourglass	Lateral buckling in mid spar followed by crushing 10½ inches inside load point.	1	15,320	12.1	.950	.437	69,800	4,742
18B	13	do	Crushing of compression flange 11 inches from load point.	1	19,050	13.6	1.000	.343	80,100	9,135
19A-1	5J, 13	Tube framework	Compression of web diagonals between end and side loads.	1	14,250	8.5	.500	.242	31,950	8,320
19A-2	5J, 13	do	Lateral buckling 8½ inches inside load points.	1	14,580	8.6	.750	.350	33,360	8,260
19B	7	do	Compression of web diagonals between side load points and ends.	1	18,440	9.98	1.000	.202	31,420	13,650

TABLE 1.—*Data from tests on complete spars*—Continued

DURALUMIN SPARS—Continued

Spar 1	Figure 2	Type 3	Failure 4	Number supports 5	Ultimate load 6	Weight of 7 feet 7	δ at ultimate P	δ at 10,000 8	$\frac{EI_{xx}}{1000}$ 9	$\frac{EI_{yy}}{1000}$ 10
20A	7	Hexagon cell box	Crushing of end fitting	1	19,740	14.66	1.15	0.308	93,900	9,310
20B	9	do	Lateral buckling	1	23,510	14.56	1.30	.375	91,400	8,750
21A	7	Box	Buckling of web followed by tearing away from compression chord and rivets.	1	20,180	14.84	.85	.292	93,400	9,500
21B	7	do	Web tore away from compression chord 9¾ inches inside load point.	1	20,980	14.25	1.30	.307	87,400	9,550
21C	9	do	Buckling of web followed by lateral buckling of compression flange.	1	20,450	14.50	.85	.268	100,100	10,130
21D	9	do	Buckling of compression flange 6¼ inches from center.	1	21,920	14.62	.95	.213	97,500	10,550
22A	7	Channel truss	Rivets connecting webs to chords near ends	1	18,450	13.81		.404	74,350	13,270
22B		do	Shear of rivets holding stiffener to compression chord channel near center followed by buckling of channel at same point.	1	19,800	13.57	1.75	.366	82,900	13,080
23A	9	Dumb-bell	Lateral buckling of compression flange followed by compression failure 15 inches from center.	1	17,580	12.94	1.00	.380	68,100	7,090
23B-1	9	do	Lateral buckling	1	15,600	11.73	.90	.430	62,700	6,705
23B-2	9	do	do	2	15,760	11.73	.90	.405	62,700	6,705
			STEEL SPARS							
30A	6R	Channel truss	Crushing of compression flange 8½ inches inside load points.	2	20,800	13.7	1.050	0.320	89,000	5,780
30B	6R, 13	do	Crushing of compression flange at weld 5½ inches inside load point.	2	17,230	12.3		.315	93,100	5,820
31A	11	Tube truss	Lateral buckling of compression flange 26 inches inside load points.	1	14,125	12.7	.600	.260	72,400	5,280
31B	4C	do	Lateral buckling	2	16,700	12.4	.900	.328	74,100	5,380
31C-1	9	do	Bending of end fittings	1	20,480	18.52	.75	.252	97,100	12,090
31C-2		do	Buckling of compression flange in bay next inside of load point.	1	22,120	18.52	.90	.280	97,100	12,090
32A-2	7	do	Crushing of compression chord 3¼ inches from center.	2	17,250	14.28	1.00	.365	89,000	6,090
32B	7	do	Crushing of compression adjacent to end fitting.	2	19,660	14.40	1.10	.364	87,000	6,390
			COMBINATION SPARS							
40A	5L, 60	Tube truss	Local buckling of compression flange at center	2	15,270	11.1	1.250	0.472	45,500	5,010
40B	9	do	Excessive deflection in plane of side loads	1	18,300	18.21	1.80	.445	54,300	18,700

TABLE 2.—*Results of tests on minor parts*

Spar	E_c	E_t	E_w	PL_c	PL_t	PL_w	U_c	U_t	U_w	YP_c	YP_t
10A											
10B		10,760			22,570						
10C								59,600			
10D								54,380			31,410
10E								54,160			30,075
11A							66,300	55,325		42,650	33,440
11B	10,750			21,450			64,400				
12A			10,320			25,720					
12B			10,520			19,920			51,600		
13A			10,300			21,000			51,450		
13B			10,310			19,950			53,300		
14A		11,070	10,500	22,950		18,830			54,800		
14B		10,970	10,550	22,200		24,480		60,000	58,500		
14C		10,960		25,390				58,200	63,300		
14D		10,895			28,050			49,875			
15A								62,720			
16A								59,675		33,400	
16B								54,500			31,775
16C								55,800			
16D								54,200			
16E								44,840			
16F								42,850			
16G								55,350			
16H								58,550			
17A		10,297			28,730			58,100			
17B		11,949			27,670			59,750			
18A		11,494			21,340			58,900			
18B		10,578			23,150			59,450			
19A								58,850			
19B								62,200			
20A		10,706			18,340			63,300			
21A	10,575			23,315			61,860	55,000			
22A		11,260			27,800						
22B		10,880			24,500			62,390			
30A								59,400			
30B								165,400			147,400
30B								160,700			147,600
31A	28,333			52,080			109,600	156,650			141,100
31B	28,111			52,000			97,300				
32A							102,000				
32B							175,700			74,000	
40A	10,271			25,000			58,700				

DESCRIPTION OF SPARS

14. The general features of the spars tested are shown in the pictures, Figures 4 to 13. The reference in Table 1 to the figures in which the separate spars are shown do not always refer to illustrations of the specific spar but may refer to a similar one of practically the same design. When the number of the spar is shown in the figure, the reference is to the figure number only, as "7" is a reference to Figure 7. If the spars in the photographs are designated by letters, the proper letter follows the figure number in the reference of Table 1. Thus "4C" is a reference to spar C shown in Figure 4. In a number of cases the spars incorporate special design features not shown in the figures and which will be described later in the discussion of the test results. A brief description of the spars tested is given below:

1A and 1B. Two plain rectangles of spruce. For small depths this section shows up very well on account of the large form factor, and it was desired to learn its relative efficiency when the allowable depth is as great as was allowed in this series of tests. The failures of these spars are shown in Figure 10.

2A. A conventional wood box with unequal spruce flanges and two-ply spruce webs. As with the other spars of type 2, the end pin was placed at the neutral axis of the section. This spar is shown in Figure 11.

2B and 2C. Spar 2A showed so much excess strength that two more conventional boxes were designed and tested. More care was taken in design in order to prevent excess strength, but there was no change in type; only a change in detail dimensions. Spar 2C is shown at *N* in Figure 5.

3A. This was the first of a series of spruce T-box spars tested to determine the effect of cambering a spruce box. It differs from the conventional box in that strips of spruce nicknamed "ears" are glued to the plywood webs on each side of the compression flange. These ears were tapered from the points of application of the side loads to a point about three inches from the end fittings, where they were cut off. As the pins through which the axial load was applied were located on the neutral axis of the end cross-section, the result of this arrangement was to introduce a beneficial eccentricity of application of the axial load in that part of the spar in which the stresses were large. As the top and bottom surfaces of the spar were parallel planes but the locus of the neutral axes of the cross sections was a curved surface, the arrangement might be described as a hidden camber. On account of this feature the spars of type 3 are classified as eccentrically loaded spars. The results of the tests on them are not considered as representative of what can be done in practice in box spar design so much as illustrations of the practicability and effectiveness of the principle of camber as applied to box spars. This type of spar might be described as a double I spar of the general type used in the Curtiss O-1 and other designs except that the ears are omitted on the tension flange. This particular spar is not shown in any of the figures in this report, but it was very similar to spar 3E shown in Figure 7.

3B. Spar 3A failed under a relatively low load by crushing of that part of the compression flange located between the plywood webs about midway between one end and the adjacent side load. As the failure was believed to be due to failure of the glue joint to carry load into the ears, spar 3B was made with the ears extending to the end fittings instead of stopping 3 inches short of that point. In other respects it was the same as 3A.

3C. This spar differed very little from 3B, the detail dimensions being varied a little in the hope of obtaining a spar of this type that would carry the required load without failure of the plywood web to transmit load into the ears.

3D. The chief feature of detail design of this spar was that a direct connection through a glued joint was provided between the ears and the central portion of the compression flange. This feature made it possible to impose a fair share of the load on the ears, and the spar failed by horizontal shear on the webs, between the end fitting and the side load.

3E and 3F. Spar 3F was a T-box, differing from 3D mainly in that web stiffeners were provided where the shear was large. It failed by a combination of shear and bending at the end fitting. The strength-weight ratio developed, 1925, is the largest yet attained. The results of the test on this type of spar show that such design refinements as camber can be applied to wood construction just as easily, if not more easily, than to metal construction, and that the resulting increases in strength-weight ratio will be as great. Spar 3E was similar to 3F, but the webs were made of inferior material. It was not as stiff as 3F and failed under 17,740 pounds by shearing of the webs and crushing of the compression flange about 15 inches from the end pin. As the weight was 10.07 pounds, the strength-weight ratio was only 1760. The EI values of this spar are unknown, as material was planed off the compression flange just before the test, the previous test of spar 3F indicating that the original flange would be stronger than the inferior plywood webs. Owing to the relatively low strength-weight ration attained and the lack of information regarding EI, the results of the test of this spar are not tabulated. The T-box spar shown in Figure 7 is 3E.

4A, 4B, 4C, and 4D. These are all conventional spruce I-beams differing slightly in proportion, but all routed out of a single stick of wood. The failure of spar 4A is shown in Figure 10, while section of 4B and 4C are shown at I and H respectively in Figure 5.

4E. A conventional I-beam differing from the other four only in that, instead of being routed out of a single stick of wood, it was made up of three pieces glued together. One piece formed the web and the other two the flanges.

DURALUMIN SPARS

10A. A duralumin box composed of two flat plates for the cover plates and two channels formed from sheet material for the webs. The webs were stiffened by vertical channels formed from sheet, one leg of these channels being riveted to each web. This spar is shown at *G* in Figure 4 and its failure in Figure 11.

10B. This spar differed from 10A only in that the rivets connecting the webs to the cover plates were spaced more closely. Its failure is shown in Figure 11.

10C. In this spar an attempt was made to improve the efficiency of the design by increasing the thickness of the cover plates and decreasing that of the webs. The stiffeners were vertical but lightened by flanged holes. This spar is shown in Figure 10.

10D. The dimensions of this spar were the same as those of 10C except that the web stiffeners were arranged in the form of the web of a Warren truss. It is shown in Figure 10.

10E. This spar was like 10C except that flanged lightening holes were put in the webs. It also is shown in Figure 10.

11A. The chords of this spar were round duralumin tubes. The webs were rather heavy flat plates with rather large unflanged lightening holes. The connection between webs and chords was made by small bolts passing through the chord tube and both web plates. Vertical channels were used between adjacent lightening holes as web stiffeners. The lightening holes were circular near the ends of the specimen where the shear is large, and oblong in the central portion where the shear is small. The failure of this spar is shown in Figures 10 and 11.

11B. This spar differed from 11A only in that screws similar to wood screws were used to connect the webs to the chords. It is shown at *B* in Figure 4 and in Figure 11.

12A and 12B. Extruded duralumin I-beams with curved flanges and bulbs at the free edges as shown at *E* in Figure 4. The failures of these spars are shown with the spars still in the testing machine in Figures 12 and 13. The same spars are shown after being taken out of the testing jig in Figure 12.

13A and 13B. Extruded duralumin I-beams with beveled flanges as shown at *D* in Figure 4. The failure of 13A is shown in Figure 12 with the spar still in the testing machine, while both spars are also shown in Figure 12 as they were after being removed from the test jig. Both type 12 and type 13 were shallower than the 6¼ inches allowed by the specification. This was due to the fact that at the time they were ordered manufacturing facilities did not permit the extrusion of a deeper section.

14A. This spar was a duralumin plate girder made up of a duralumin sheet web stiffened by small vertical angles and bulb T extruded flanges. The vertical leg of the T was slotted so the web plate could be placed on the plane of symmetry. This spar is shown at *F* in Figure 4.

14B. This spar differed from 14 A in that, instead of slotting the extruded T section to receive the web plate, that plate was riveted to one side of the vertical leg. In order to keep the weights of these two spars as nearly the same as possible, the side of the T was planed down an amount equal to the thickness of the web. This spar is shown at *A* in Figure 4 and its failure in Figures 12 and 13.

14C. This spar was of the more conventional plate girder type, the flanges being composed of two angles. The two angles used, when taken together have the same cross section as the bulb T used for spars 14A and 14B. Spar 14C is shown at *K* in Figure 5.

14D. A plate girder similar to 14B with its eccentrically located web plate, except that the amount of material in the bulbs of the extruded T flanges was increased, and the vertical leg of the T was not recessed for the web plate.

15A. A duralumin channel Warren truss. The chord members were single channels with spacers tying the free edges together to prevent local failure. Each web member was composed of two channels facing each other. About at mid-height of the beam the free edges of these web channels were connected by small plates to prevent local buckling. All of the channels used in this spar had a ratio of width of back to length of leg of about unity. The connections were riveted. Probably through oversight, the depth of this spar was made 6⅝ inches instead of the 6¼ inches specified. This difference resulted in the spar giving better test results than if this error had not been made, and, therefore, the results for this spar are not directly comparable with those of the others. This spar is shown in Figure 12.

16A. Another duralumin channel Warren truss differing in many details from 15A. Single channels were used for both chord and web members, and the ratio of width of back to length of leg was about 2 in all cases. On account of the short legs, no spacers were used to tie the free edges of the channels together and prevent local buckling. This spar is shown at *P* in Figure 6 and its failure in Figure 13.

16B. A channel truss design very similar to 16A. The width of back of the channels forming the chords was about one-fourth inch greater than those of 16A while the widths of their sides

was about one-eighth inch less, and the thickness of the sheet from which they were constructed was slightly less. As a result a somewhat greater moment of inertia was obtained coincidentally with a small decrease in sectional area. The weight, however, was a little greater, due probably to heavier web members and connections. Spars 16B to 16H are all shown in Figure 8.

16C. The chords of this spar had backs with the same width as 16B and sides with the same width as 16A. The thickness of the material was practically the same as for the other two spars.

16D. The width of back of the compression chord channels of this spar was even less than for 16A, the width of side was the same as for 16A and 16C, and the material from which they were made was of about 25 per cent heavier gauge. The tension chord channel was about a tenth of an inch narrower of back, and of the same width of side, but the thickness of the material used was only about 0.6 that in the compression chord.

16E. The dimensions of the chord channels were practically the same as for 16D except that the sides of the compression chord were made wider and of the tension chord narrower.

16F. The chords of this spar had the widest backs and the narrowest sides of all of type 16 that were tested. The thickness of the material was the same as in spars 16A and 16B. Both chords were of the same size, and the backs of both were stiffened with longitudinal corrugations.

16G. The chords of this spar were of the same size as those of 16C except that the backs of the channels were stiffened by a longitudinal corrugation.

16H. This spar was like 16G except that the chords were additionally stiffened by bending the free edges of the sides inwards.

17A. A duralumin trussed web dumb-bell spar. It is shown at *S* in Figure 6 and in Figure 13. It should be noted that the web of Warren truss type is composed of members of stamped sheet.

17B. This spar was like 17A except that the web members were composed of duralumin tubes flattened at the ends. It is shown at *T* in Figure 6 and in Figure 13.

18A. A duralumin hour-glass type spar similar to some of the steel spars developed in England. It is shown at *M* in Figure 5 and in Figure 11.

18B. As 18A failed by lateral buckling at a relatively low load, the second hour-glass spar, 18B, was made wider, but otherwise the same. Its failure is shown in Figure 13.

19A. A framework of duralumin tubing. The compression chord was composed of two round tubes connected by a shallow channel so the whole chord would act as a unit in resistance to lateral buckling. The tension member was composed of a single round tube. The webs were composed of a number of small round tubes pinned to the chord tubes. These web members were in four planes, each compression chord tube being connected to the tension chord by two sets of web members. The construction of this spar is shown at *J* in Figure 5 and also in Figure 13. As this spar was cambered by assembling it in a slightly arched shape, and also had eccentrically located pins in the end fittings, it is classed among the eccentrically loaded spars and the test results are not comparable with those in the other two groups. In the first test the compression web members near one end fitting failed under a low load and the spar was returned to the manufacturer for repair. Heavier web members were put in and the spar tested again. The spar in the form first tested is referred to in the tables as 19A–1, and the repaired spar as 19A–2. The photograph of this spar in Figure 13 was taken directly after the first test.

19B. The chief differences between spars 19A and 19B are that in the latter, the main tubes are somewhat larger, and the channel connecting the two tubes of the compression chord of 19A are replaced in 19B by a latticing of small round tubes similar to the latticing connecting the two chords. This spar is shown in Figure 7.

20A. Spar 20A was a duralumin box with the chords made of two sheets forming a hexagonal cell. The webs were of corrugated duralumin sheet, and the corrugations were vertical. In the first test the rivets in the end fittings failed. These rivets were replaced by machine screws and in a second test the spar failed in the webs near the end fittings. The data given below are from the second test.

20B. As spar 20A failed in the end fittings, 20B was made of the same size, except for the fitting design. This spar is shown in Figure 9.

21A. Spar 21A was a development of type 10, differing from that type in several important details. Instead of a single flat cover plate for each chord, two plates were used, the inner one flat and the outer with the free edges turned down to provide added stiffness. Also the tension chord was made lighter than the compression chord. The web, instead of being stiffened by channels connecting their inner faces, as in type 10, were stiffened by fairly large vertical corrugations spaced about 6 inches apart. This spar is shown in Figure 7.

21B. This spar differed from 21A in that the compression chord was made a little lighter by decreasing the thickness of the inner plate and the tension chord by omitting the inner plate but increasing the thickness of the outer plate. This spar is also shown in Figure 7.

21C. Like 21B except that the webs were stiffened by internal channels like those used in type 10 alternating with small angles riveted to the outer surfaces. This spar is shown in Figure 9.

21D. Like 21C except that only the channel stiffeners were used. The spar is shown in Figure 9.

22A. Spar 22A was a duralumin channel truss very similar to the spars of type 16. The chief differences were that the tension chord was made lighter than the compression (though this was true of 16D and 16E). The back of the compression chord channel was reinforced by a longitudinal strip of sheet riveted to the center, the web members were attached by two large rivets to each side of the chord member instead of several small rivets and the tension web members were made up of pairs of small channels, one on each side of the spar instead of all web members being made of single channels connected to both sides of the chord channels. In the first test the free edges of the compression web members buckled due to the added compression caused by the eccentric application of the load involved in the design of the connections to the chords, and had to be stiffened by riveting small angles to these free edges. Spar 22A is shown in Figure 7.

22B. This spar was like 22A, except that the compression web members were made with the free edges turned in to provide stiffness and avoid the weakness developed in 22A.

23A. A duralumin dumb-bell type with the web made of sheet duralumin corrugated longitudinally. The spar is shown in Figure 9.

23B. This spar was like 23A except that the web was made of flat sheet reinforced by D type stiffeners in the form of a Warren truss web. This spar is shown in Figure 9.

STEEL SPARS

30A. A steel channel Warren truss. This spar was very similar to spar 16A, but constructed of heat-treated alloy steel sheet. On account of the greater density of steel this spar was much narrower than 16A and had such a low value of EI_{yy} that it was tested with two lateral supports. It is shown at *R* in Figure 6.

30B. This spar was like 30A except that the joints were made by welding after heat treatment instead of being riveted. It is shown at *Q* in Figure 6 and its failure in Figure 13.

31A. A welded chrome-molybdenum steel tube Warren truss. On account of the small allowable total depth the chord members were made of elliptical tubing, though the web members were made of round tubing. This spar is shown at *C* in Figure 4.

31B. In manufacturing spar 31A the manufacturer did not make sufficient allowance for shrinkage of the spar after welding, and the first spar constructed was found to be more than an eighth of an inch below the specified total depth. In spite of this defect it was tested, being designated 31B.

31C. Spars 31A and 31B were insufficiently stiff laterally, so that much larger tubes were used in 31C. Otherwise it was like the other two spars. It is shown in Figure 9.

32A and 32B. Spars 32A and 32B were welded chrome-molybdenum steel tube trusses very similar to those of type 31. The chords were of elliptical tubing and the webs of round tubing. The chief detail of interest in their design was that the tension web members were carried through the chord member tubes and were welded to both surfaces of the latter. Spar 32A was made of tubing as received from the mill without heat treatment. It failed by crushing of the compression chord near a weld about 3½ inches from the center of the length of the spar. The tubes for spar 32B were heat treated to 150,000 pounds per square inch tension before welding. This spar failed by crushing of the compression chord adjacent to the end fitting. Both of these spars were tested with two lateral supports. An attempt was made to test spar 32A with a single lateral support, but the compression flange began to buckle at about 12,000 pounds axial load, and the test conditions were changed. The spars are shown in Figure 7.

COMBINATION STEEL AND DURALUMIN SPARS

40A. A spar similar to type 31, the chief difference being that the chords were made of elliptical duralumin tubing. As dural chords can not be welded to steel tube webs, the web members were welded to sheet-steel saddles and the chord members pinned to these saddles. This spar is shown at *L* in Figure 5 and also at *O* in Figure 6.

40B. Like 40A except that larger tubes were used for the chords. This spar is shown in Figure 9.

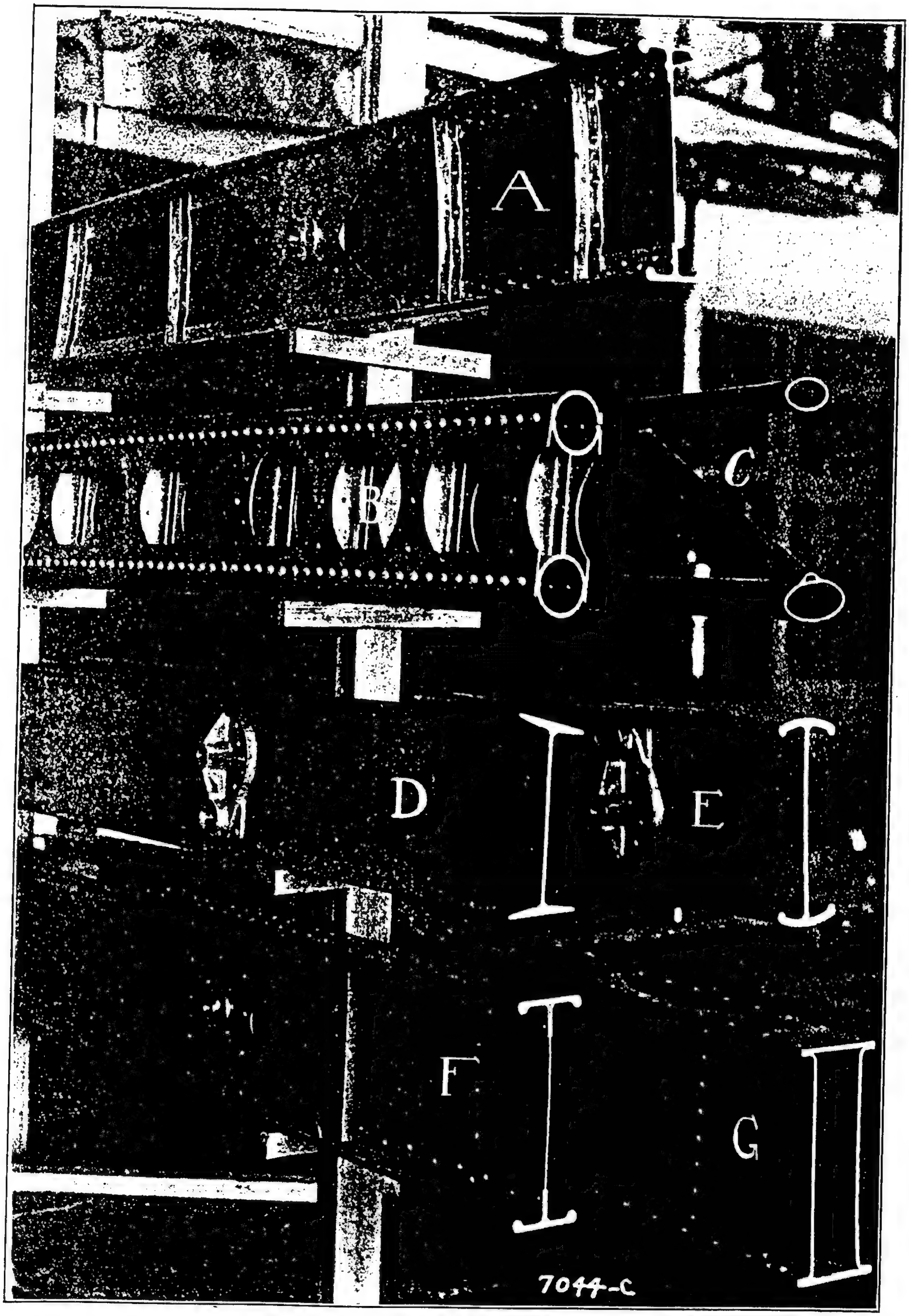

FIG. 4

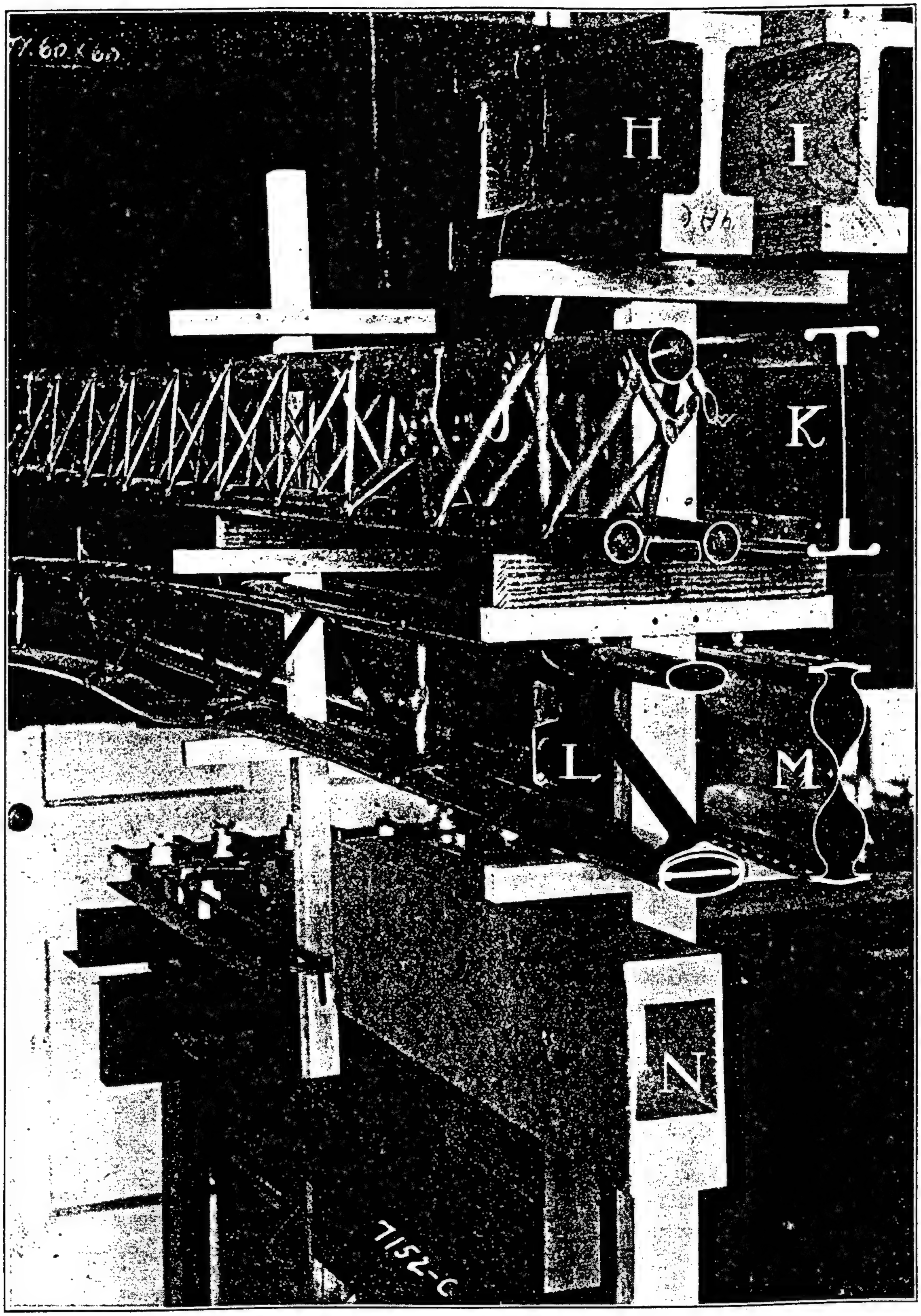

FIG. 5

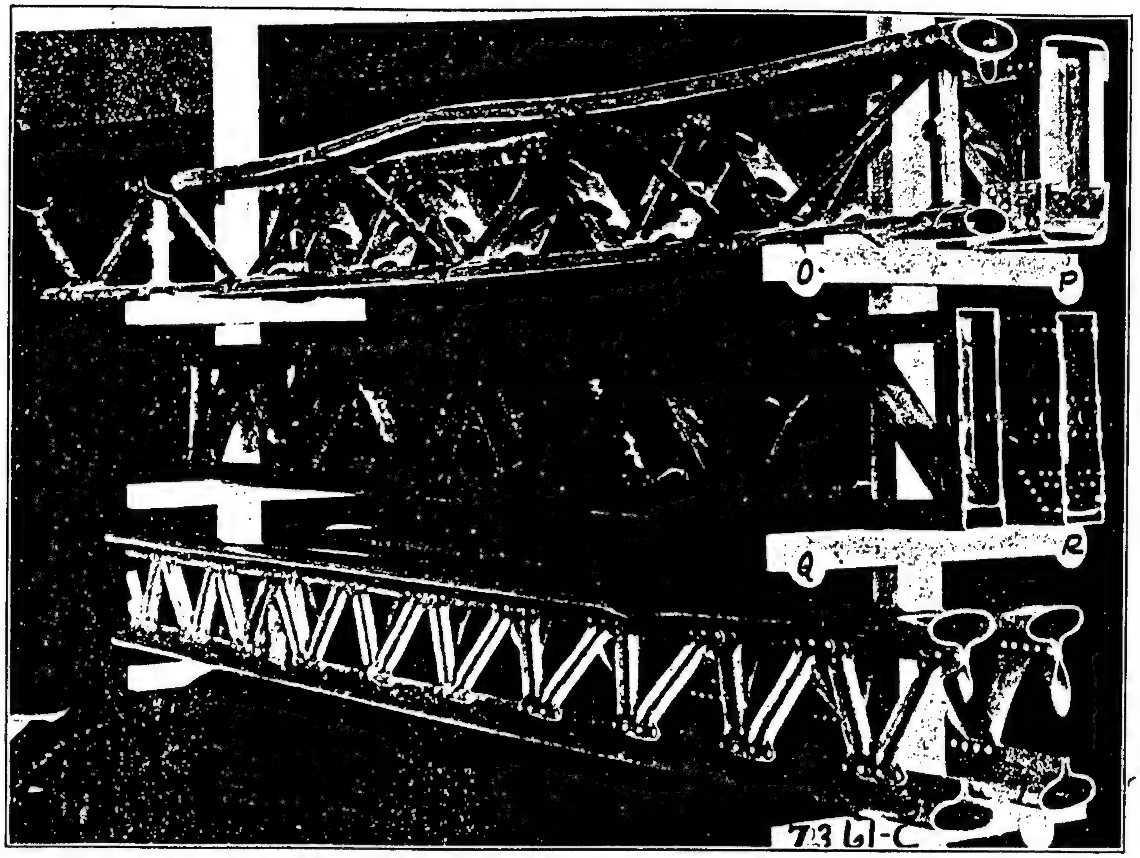

FIG. 6

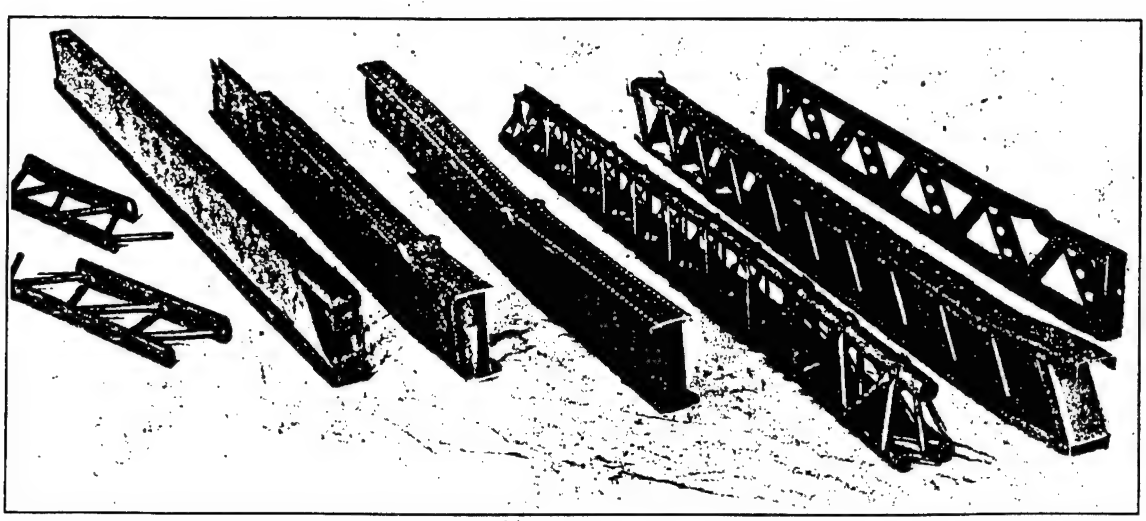

FIG. 7

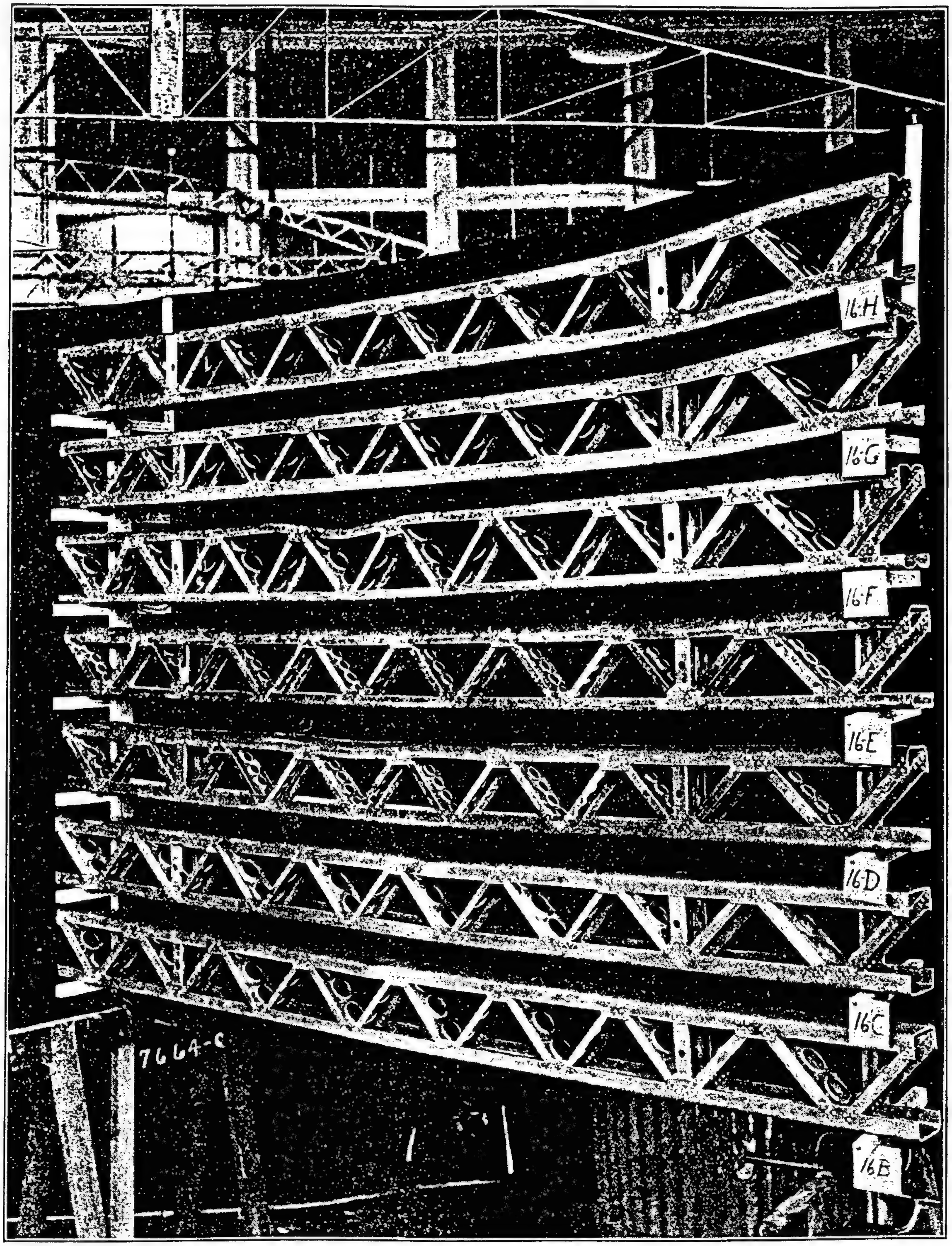

FIG. 8

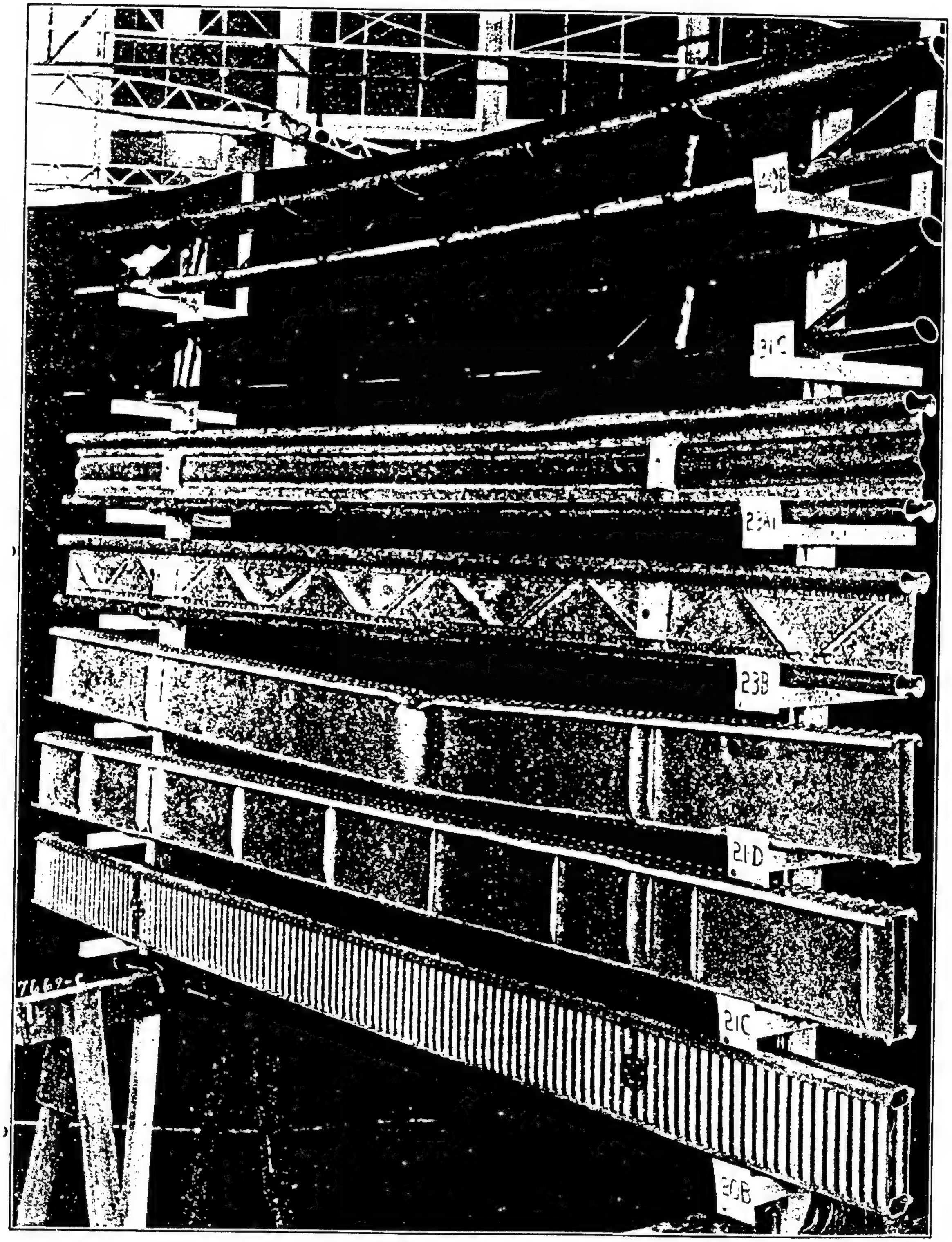

FIG. 9

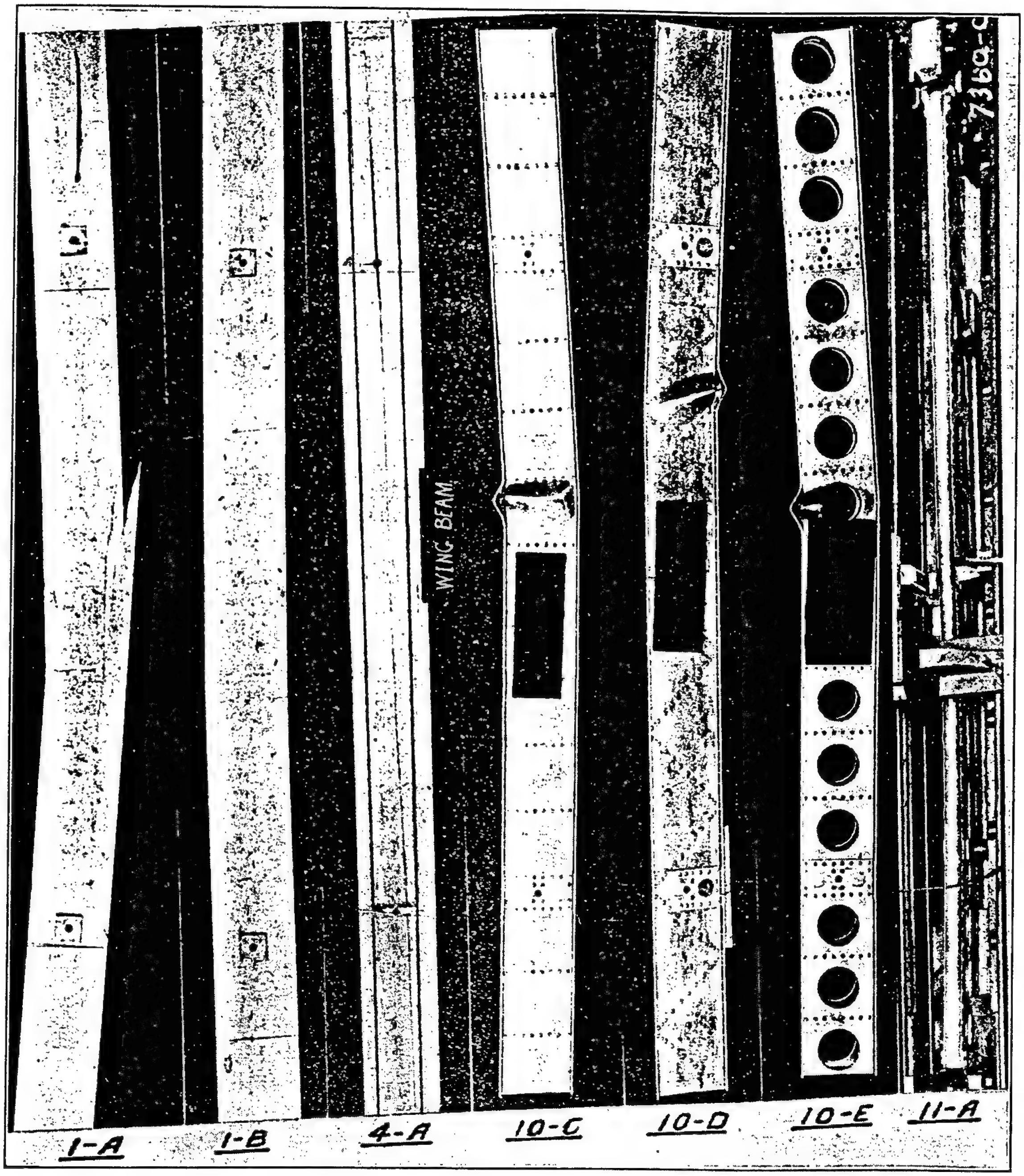

FIG. 10

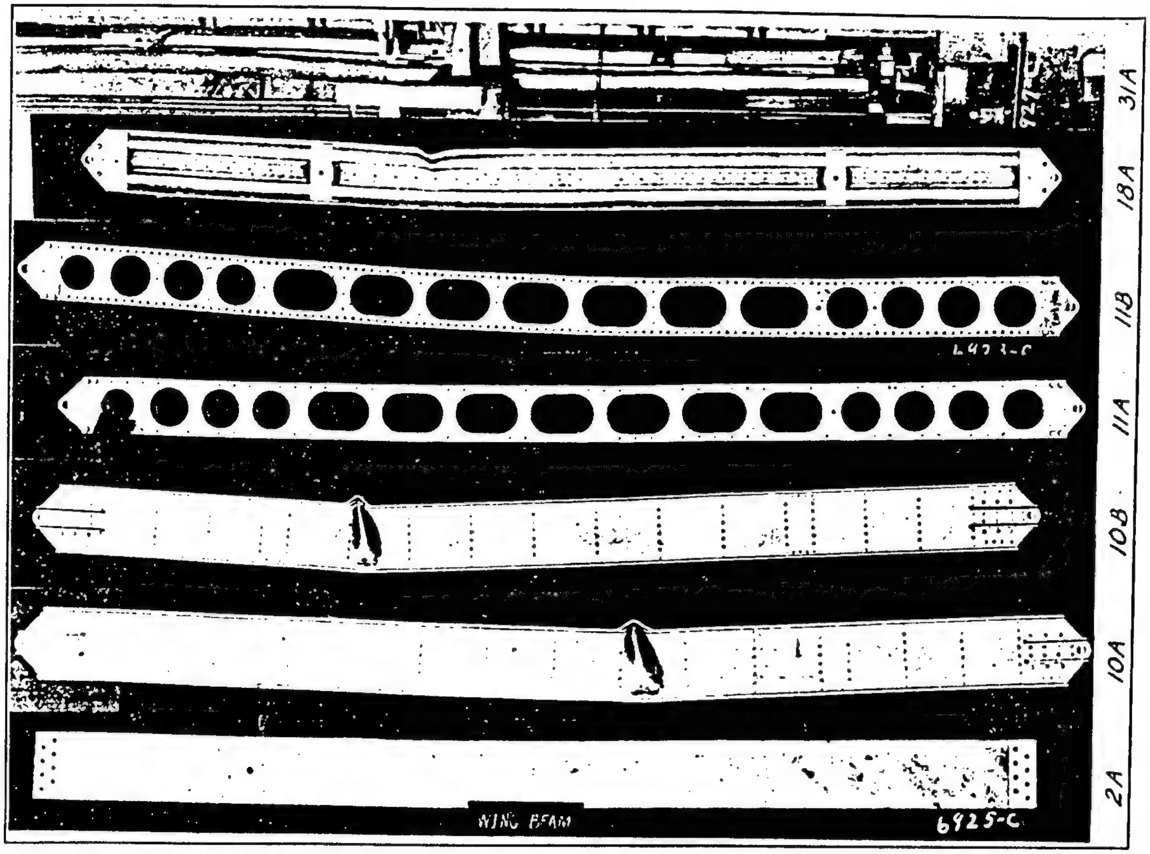
31A
18A
11B
11A
10B
10A
2A
WING BEAM
6925-C

Fig. 11

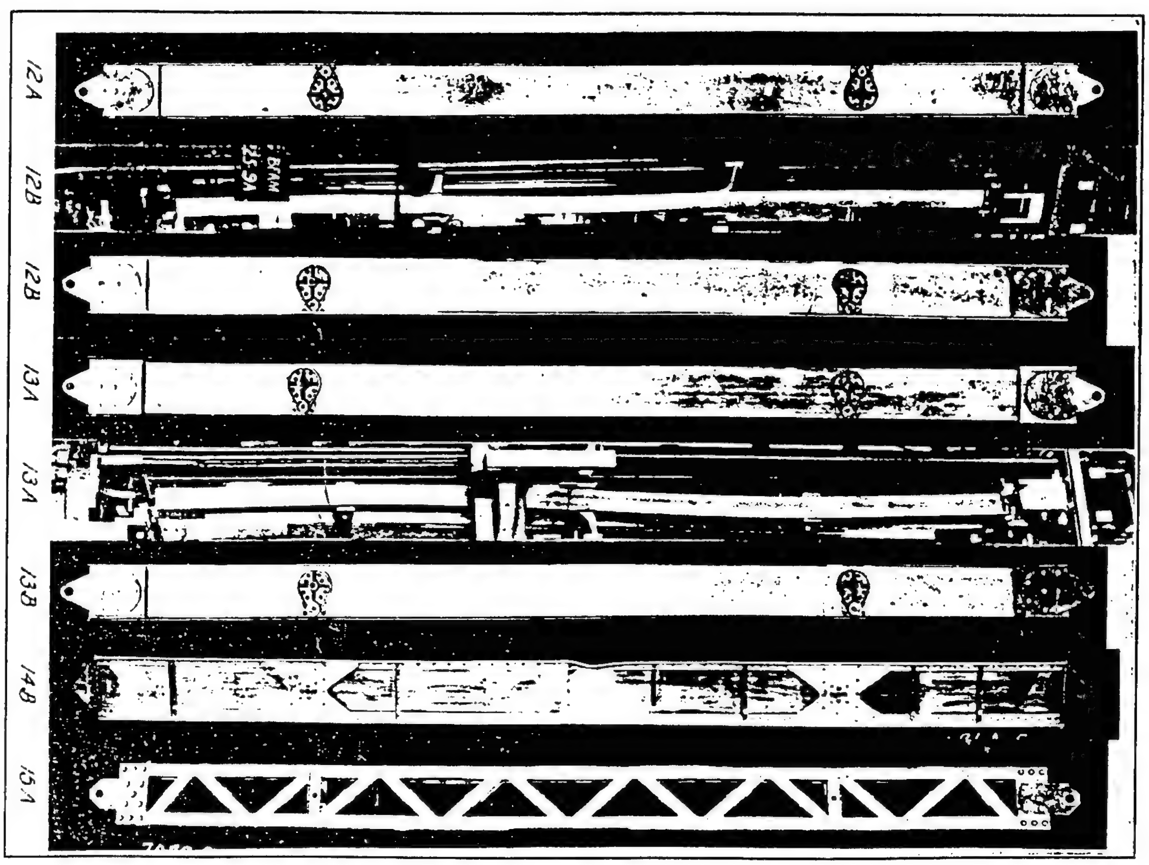
12A
12B
12B
13A
13A
13B
14B
15A

Fig. 12

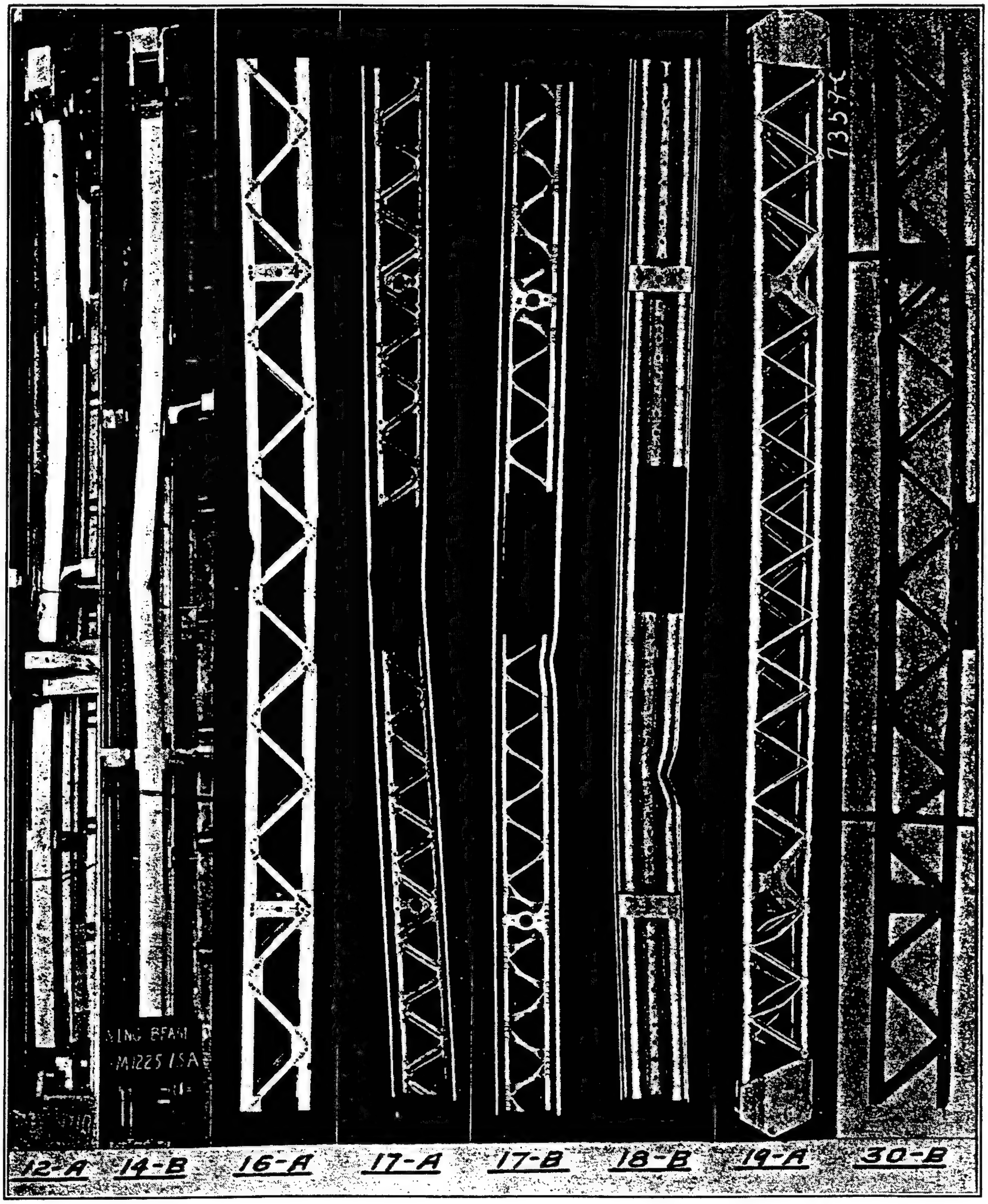

Fig. 13

GEOMETRICAL PROPERTIES OF SPARS

15. The geometrical properties of the cross sections of the metal spars are given in Table 3. The actual dimensions are not given on account of lack of space, but wherever there might be any question as to how the values given in Table 3 were obtained, the method used is explained in paragraph 16. The wood spars are not represented in this table as they were designed in accordance with standard methods and present no special feature or problems.

The column headings of Table 3 have the following meanings:

Column	Heading	Significance
2	A_c	Sectional area of compression chord.
3	A_t	Sectional area of tension chord.
4	A_w	Sectional area of web (omitted in the case of trusses).
5	A	Sectional area of spar; the sum of columns 2, 3, and 4.
6	I_x	Moment of inertia of spar about minor of XX axis.
7	I_y	Moment of inertia of spar about major or YY axis.
8	c_c	Distance from centroid of section to most distant fiber of compression chord.
9	c_t	Distance from centroid of section to most distant fiber of tension chord.
10	y_c	Distance from centroid of section to centroid of compression chord.
11	y_t	Distance from centroid of section to centroid of tension chord.
12	h	Effective depth of spar; the sum of y_c and y_t.
13	ρ_x	Radius of gyration of section about minor or XX axis.
14	ρ_y	Radius of gyration about major or YY axis of section.
15	ρ_{cx}	Radius of gyration of compression chord about an XX axis through its centroid.
16	ρ_{cy}	Radius of gyration of compression chord about a YY axis through its centroid.

16. In computing the geometrical properties listed in Table 3 the procedure used in doubtful cases was as stated below:

TABLE 3.—*Geometrical properties of spar sections*

DURALUMIN SPARS

Spar 1	A_c 2	A_t 3	A_w 4	A 5	I_x 6	I_y 7	c_c 8	c_t 9	y_c 10	y_t 11	h 12	ρ_x 13	ρ_y 14	ρ_{cx} 15	ρ_{cy} 16
10A	0.3135	0.3135	0.844	1.471	9.18	1.197	3.125	3.125	3.0705	3.0705	6.141	2.498	0.902	0.03145	0.830
10B	.3135	.3135	.844	1.471	9.18	1.197	3.125	3.125	3.0705	3.0705	6.141	2.498	.902	.03145	.830
10C	.359	.359	.691	1.409	9.45	1.126	3.125	3.125	3.0625	3.0625	6.125	2.590	.894	.1141	.830
10D	.359	.359	.691	1.409	9.45	1.126	3.125	3.125	3.0625	3.0625	6.125	2.590	.894	.1141	.830
10E	.350	.359	.417	1.135	9.27	.9045	3.125	3.125	3.0625	3.0625	6.125	2.858	.8925	.1141	.830
11A	.614	.614	--------	1.228	6.584	.512	3.125	3.125	2.250	2.250	4.50	2.316	.646	.545	.602
11B	.614	.614	--------	1.228	6.584	.512	3.125	3.125	2.250	2.250	4.50	2.316	.646	.545	.602
12A	.550	.550	.600	1.700	8.425	.433	2.800	2.800	--------	--------	--------	2.226	.504	--------	.626
12B	.550	.550	.600	1.700	8.425	.433	2.800	2.800	--------	--------	--------	2.226	.504	--------	.626
13A	.500	.500	.600	1.600	7.881	.391	2.986	2.702	--------	--------	--------	2.219	.494	--------	.625
13B	.500	.500	.600	1.600	7.881	.391	2.986	2.702	--------	--------	--------	2.219	.494	--------	.625
14A	.512	.512	.230	1.254	8.980	.407	3.125	3.125	2.894	2.894	5.788	2.676	.570	.253	.631
14B	.501	.501	.212	1.214	8.751	.381	3.125	3.125	2.890	2.890	5.779	2.685	.560	.256	.616
14C	.652	.652	.212	1.516	10.859	.445	3.125	3.125	2.826	2.826	5.652	2.676	.542	.309	.584
14D	.633	.633	.211	1.477	10.771	.581	3.150	3.150	2.862	2.862	5.724	2.700	.628	.300	.676
15A	.580	.580	--------	1.160	9.206	.854	3.3125	3.3125	2.776	2.776	5.552	2.817	.858	.429	.858
16A	.630	.630	--------	1.261	9.933	1.657	3.125	3.125	2.781	2.781	5.562	2.807	1.124	.3780	1.046
16B	.623	.623	--------	1.247	10.274	1.834	3.150	3.150	2.850	2.850	5.701	2.870	1.213	.3422	1.212
16C	.675	.675	--------	1.350	10.902	2.039	3.155	3.155	2.815	2.815	5.631	2.842	1.229	.3810	1.228
16D	.728	.445	--------	1.173	8.755	1.340	2.487	3.813	2.116	3.461	5.577	2.732	1.069	.3726	1.066
16E	.776	.389	--------	1.166	8.233	1.335	2.291	3.969	1.863	3.718	5.576	2.657	1.070	.4255	1.090
16F2	.594	.594	--------	1.188	10.208	1.793	3.140	3.140	2.922	2.922	5.845	2.931	1.229	.2300	1.228
16G	.677	.677	--------	1.355	10.770	2.011	3.145	3.145	2.795	2.795	5.590	2.819	1.218	.3700	1.218
16H	.693	.693	--------	1.386	11.034	2.100	3.145	3.145	2.797	2.797	5.595	2.822	1.231	.3684	1.231
17A	.500	.500	--------	1.000	6.767	.542	3.156	3.156	2.570	2.570	5.140	2.601	.736	.402	.736
17B	.500	.500	--------	1.000	6.317	.533	3.125	3.125	2.498	2.498	4.995	2.513	.730	.278	.736
18A	.285	.285	.749	1.319	7.067	.533	3.094	3.094	2.90	2.90	5.80	2.315	.636	--------	.636
18B	.382	.382	.670	1.434	7.917	.982	3.063	3.063	2.88	2.88	5.76	2.350	.828	--------	.828
DURALUMIN SPARS															
19A	0.509	0.263	--------	0.772	4.358	1.097	2.234	4.016	1.685	3.266	4.951	2.376	1.192	0.280	1.421
19B	.554	.228	--------	.782	3.683	1.508	1.988	4.062	1.363	3.312	4.675	2.170	1.388	.416	.616
20A	.710	.710	--------	1.420	10.333	.866	3.125	3.125	2.729	2.729	5.458	2.698	.781	.296	.781
20B	.675	.675	--------	1.350	10.083	.846	3.125	3.125	2.716	2.716	5.432	2.733	.792	.296	.792
21A	.627	.468	0.494	1.589	11.374	1.034	2.862	3.438	2.735	3.335	6.070	2.675	.808	.080	.864
21B	.609	.393	.494	1.496	10.209	1.021	2.713	3.537	2.579	3.432	6.011	2.612	.827	.096	.886
21C	.609	.393	.494	1.496	10.209	1.021	2.713	3.537	2.579	3.432	6.011	2.612	.827	.096	.886
21D	.609	.393	.494	1.496	10.209	1.021	2.713	3.537	2.579	3.432	6.011	2.612	.827	.096	.886
22A	.684	.443	--------	1.127	9.013	1.219	2.548	3.752	2.265	3.492	5.757	2.828	1.040	.303	1.008
22B	.666	.553	--------	1.219	10.011	1.314	2.854	3.396	2.601	3.130	5.730	2.866	1.038	.266	.997
23A	.460	.460	.320	1.240	5.932	.582	3.125	3.125	2.380	2.380	4.760	2.190	.685	.518	.793
23B	.460	.460	.1408	1.061	5.596	.578	3.125	3.125	2.380	2.380	4.760	2.296	.737	.518	.793
STEEL SPARS															
30A	0.208	0.208	--------	0.416	3.5345	0.1934	3.125	3.125	2.906	2.906	5.811	2.915	0.683	0.2505	0.683
30B	.208	.208	--------	.416	3.5345	.1934	3.125	3.125	2.906	2.906	5.811	2.915	.683	.2505	.683
31A	.277	.192	--------	.469	3.229	.184	2.726	3.524	2.163	3.118	5.281	2.624	.626	.410	.707
31B	.277	.192	--------	.469	3.081	.184	2.675	3.450	2.112	3.044	5.156	2.563	.626	.410	.707
31C	.395	.262	--------	.657	4.119	.386	2.656	3.594	2.006	3.027	5.033	2.504	.768	.468	.806
32A	.259	.200	--------	.459	3.078	.191	2.825	3.363	2.250	2.913	5.163	2.590	.645	.420	.710
32B	.259	.200	--------	.459	2.977	.191	2.787	3.313	2.212	2.863	5.075	2.547	.645	.420	.710
COMBINATION SPARS															
40A	0.5458	0.2887	--------	0.835	5.379	0.420	2.407	3.893	1.827	3.453	5.280	2.538	0.709	0.402	0.768
40B	.929	.569	--------	1.497	8.141	1.741	2.731	3.320	1.758	2.871	.4629	2.332	1.080	.708	1.219

Type 10. Duralumin box.—Shown as *G* in Figure 4. In computing the properties of the sections of this type the cover plate alone was assumed to constitute the chord.

Type 11. Duralumin tube and plate spar.—Shown as *B* in Figure 4. In computing properties, that part of the web sheet between the edge of the lightening hole and the free edge was assumed to be part of the chord. Both spars were intended to be of the same section and this was assumed to be the case. The properties listed were obtained from the dimensions of spar B, and those of spar A assumed identical.

Type 12. Extruded duralumin bulb I *beam.*—Shown as *E* in Figure 4. The division between the chord and the web was assumed to be at the base of the fillet where it is tangent to the sides of the web. No attempt was made to compute the effective depth of the section or the properties dependent upon a knowledge of the location of the centroid of the chord. The moment of inertia was determined graphically. The properties of both spars were assumed the same.

Type 13. Extruded duralumin bevel I *beam.*—Shown as *D* in Figure 4. The actual section was not used in determining the geometrical properties, but an equivalent section was drawn up in accordance with the usual practice in design of wood spars, and the properties of this equivalent section assumed to be those of the actual spars.

Type 14. Duralumin plate girder.—Shown as *F* in Figure 4. In the computations, that part of the web plate between its free edge and the free edge of the vertical leg of the Tee was assumed part of the chord.

Types 15 and 16. Duralumin channel truss.—Shown as *P* in Figure 6. In computing properties allowance was made for the fillets where the sheet was bent.

Type 17. Duralumin trussed web dumb-bell.—Shown as *S* in Figure 6. The areas and other properties were obtained graphically with the aid of a planimeter. In the case of 17A, the sectional area of the chord was checked by Simpson's rule from which a slightly larger value was obtained, but the planimetered value was considered the more reliable.

Type 18. Duralumin hourglass.—Shown as *M* in Figure 5. The division between the chord and the web was assumed so that the horizontal portion of the web plate where it is riveted to the cover plate was considered as part of the chord. The properties of 18A were computed both analytically and graphically and what was considered the more reliable figure was used. The areas of 18B were computed analytically and checked graphically. The moment of inertia was found from that of 18A by proportion. The locations of the flange centroids were estimations of those quantities for the cover plate by itself.

Type 19. Duralumin tubing framework.—Shown as *J* in Figure 5. In 19A the channel connecting the two tubes in the compression chord was considered a part of that chord, but the computations of its properties are none too precise. In 19B the small tubes that replaced this channel were neglected in computing the properties of the compression chord.

Type 20. Hexagonal cell chord duralumin box.—Shown in Figure 9. None of the sheet forming the webs was considered a part of the chord, and the web is neglected in computing the total area, as the vertical corrugations were believed to prevent its carrying any appreciable portion of the axial load. The properties were obtained graphically. As an analytical check of the chord area gave a somewhat smaller value, the area tabulated may be considered as including the horizontal portion of the web sheets. The properties of 20B were taken from a report submitted by the Douglas Co., which accounts for the difference in the various properties as recorded, in spite of the fact that the two spars were intended to be of identical cross section.

Type 21. Duralumin box.—Shown in Figure 7. The part of the web plates that was turned over to permit riveting to the cover plates was considered a part of the chord. Where the inner cover plate was beveled at its edges, an equivalent rectangular section was used in computing properties. The properties of 21C and 21D were assumed to be the same as those of 21B.

Type 22. Duralumin channel truss.—Shown in Figure 7. The dimensions of the fillets at the corners of the chord members were considered in computing the properties of the section.

Type 23. Duralumin continuous web dumb-bell.—Shown in Figure 9. The properties of 23B were determined graphically. Those of 23A were determined by correcting the properties of 23B to allow for the greater thickness and corrugations of the web. The chords of the two types were assumed to be identical. None of the area of the sheets forming the webs was considered a part of the area of the chord.

Type 30. Steel channel truss.—Shown as *R* in Figure 6. The fillets at the corners of the chord member channels were neglected in computing the properties of the section.

Types 31 and 32. Welded steel tube trusses.—Shown as *C* in Figure 4. No allowance was made for extra material at the welds, and the chord members were assumed to be truly elliptical in section. In type 32 no allowance was made for loss of material where the chords are intersected by the tension web members.

Type 40. Combination duralumin and steel tube truss.—Shown as *L* in Figure 5. The chord members were assumed to be truly elliptical in section.

17. The values given in Table 3 are not as exact as the number of significant figures would imply. This is due to two main causes. The first is that the properties were computed mainly from cross-sections of the spars, cut from them after the tests. When the tests were started, it was believed that this would be the best procedure as it would insure that the actual thickness of the material in the test specimens would be used in the computation of properties rather than the nominal values used in design. Experience has shown that while the true value is used if found, there is a large chance of error in making the measurements, and that the resulting values are not necessarily more reliable than would be values worked out by the builder before submitting the spar to the division for test, particularly if the builder were required to check his intended measurements on the article as furnished. The second cause of error was the change in section resulting from the tests. This was a particularly annoying source of error in the case of spars like those

of types 17 and 23 with irregular sections. In other types, like 10 and 16, even though the spar section were deformed in the test it was not hard to reconstruct the original section. In the specification that will be used in future purchases of test spars, it is required that the builder compute all of the important geometrical properties of the spars he furnishes, checking them against the actual articles furnished, and it is believed that the data obtained in this way will be more reliable than that given in Table 3.

18. As a result of the lack of precision of the geometrical properties of the spars as given in Table 3 the unit stress values given later in this report lack precision in the same degree. It is believed that this lack of precision is not great enough to vitiate the conclusions given in the report, but its does prevent the formulation of any conclusions based upon small differences in unit stresses. Another factor which has the same practical result is the fact that the exact line of division between web and chord is often uncertain. This is very obvious in the case of types like 13 and 18, but is also true of other types. One might think that this division would be quite clear in the case of, say, type 10, but there the question arises as to whether the narrow flanges on the sheets forming the web that are needed in order to rivet it to the cover plates should be considered as part of the web or of the chord, and in the latter case just where the division line should be assumed. That the division was not taken at the proper theoretical point in the case of type 10 is shown by the fact that the computed average values of the unit stresses in the compression chords of these spars was consistently greater than the computed maximum unit stresses in these chords, and the same was true in a less pronounced degree in the cases of the other box types. This illogical result was due to the fact that the bending carried by the web was taken into account when computing the maximum unit stresses but neglected when computing the average unit stresses. That the effect was most pronounced in the case of type 10 was due to the fact that the part of the web sheets that was turned over to permit riveting to the cover plate was considered as a part of the web in the spars of that type but as a part of the chord in the cases of the other box types.

STRENGTH-WEIGHT RATIOS

19. *Strength-weight ratios.*—The test results recorded in Table 1 are not in themselves sufficient to determine the relative merit of the designs tested. For this purpose it is necessary to use quantities derived from them, though the quantities to be used to determine a satisfactory figure of merit are a subject of argument. The most important quantity used in rating the spars is the ratio of strength to weight. In this study the strength-weight ratio used was that of the axial load on the beam at failure to the weight of a 7-foot section. The same relative results could have been obtained from any of a number of similar ratios, but that chosen seemed to be the most convenient. It should be borne in mind that this figure is relative and not absolute, so that it can be used only in comparing spars designed for the same load and tested under the same conditions. The strength-weight ratios used in this report would be worthless in any attempt to compare the spars discussed with other spars designed for other conditions.

20. The strength-weight ratio by itself can not be considered as a satisfactory figure of relative merit, as the designer in selecting the type of spar to be used in a given location must consider a number of other factors, many of them nonquantitative, such as reliability, availability of material, character of failure, experience of the workmen available, applicability of the type of spar to the general design, and status of the type of design with regards to reliability of existing methods of stress analysis in determining the ultimate load. In spite of its limitations, however, it is the best single figure available for the purpose, and if the indications of the various strength-weight ratios are interpreted with judgment regarding the effects of the other factors, a very good idea of the relative merits of the various designs will be obtained.

21. The first article of a given type of design is usually poorer than later articles designed in the light of the results of the tests on the first one. The fact that a new type shows a poor strength-weight ratio, therefore, should not be taken as conclusive evidence that the type is poor, as the first test may show how the type can be greatly improved by minor modifications. In other words, care must be taken to make sure that comparisons are made between spars which represent good design in their respective types and not a good design of one type against a poor design of another type. The policy of the division has been to consider the first spar of a new type to be in the nature of a sighting shot and to make definite decisions regarding the relative merits of different types only on the basis of tests of second or later articles. In some cases this is not possible with the spars discussed in this report, as only one of a type has been tested or all spars tested were designed at the same time, before any tests had been made. In the discussions which follow an attempt will be made to take proper allowance of this fact and to make as fair judgments as the data will permit.

TABLE 4.—*Strength-weight ratios*

Spar	P	Wt	P/Wt	Standings		Relative	P/Wt Ratio	P/Wt× % Des. P	Standings		P/Wt+ √% Des. P	Standings	
1	2	3	4	5	6	7	8	9	10	11	12	13	14
SPARS TESTED UNDER DESIGN CONDITIONS													
2C	22,380	13.5	1,658	1	1	100.0	100.0	1,658	1	1	1,569	1	3
2A	24,690	15.0	1,648	2	2	99.4	99.4	1,648	2	2	1,483	10	15
20B	23,510	14.56	1,615	3	3	97.4	97.4	1,615	3	3	1,490	9	13
4C	21,320	13.3	1,603	4	4	96.7	96.7	1,603	4	4	1,554	2	4
2B	21,700	13.8	1,572	5	5	94.8	94.8	1,572	5	5	1,510	5	8
21D	21,920	14.62	1,499	6	9	90.4	90.4	1,499	6	7	1,433	24	30
16G	20,150	13.46	1,497	7	10	90.3	90.3	1,497	7	8	1,492	8	12
4B	19,750	13.3	1,485	8	11	89.6	89.6	1,466	10	11	1,494	7	11
4D	18,650	12.6	1,480	9	12	89.3	89.3	1,380	15	16	1,533	3	5
21B	20,980	14.25	1,472	10	13	88.8	88.8	1,472	8	9	1,438	22	28
4E	20,750	14.1	1,472	10	13	88.8	88.8	1,472	8	9	1,445	16	22
22B	19,800	13.57	1,459	12	15	88.0	88.0	1,444	11	12	1,466	11	16
16A	18,500	12.7	1,457	13	16	87.9	87.9	1,348	19	21	1,515	4	6
4A	19,450	13.7	1,420	14	17	85.6	85.6	1,381	14	15	1,440	21	27
15A	19,290	13.6	1,419	15	18	85.6	85.6	1,367	16	17	1,445	16	22
21C	20,450	14.50	1,410	16	19	85.0	85.0	1,410	12	13	1,394	26	33
18B	19,050	13.6	1,401	17	20	84.5	84.5	1,335	20	23	1,435	23	29
16C	18,750	13.42	1,397	18	22	84.3	84.3	1,310	22	25	1,443	19	25
10B	20,185	14.6	1,383	19	23	83.4	83.4	1,383	13	14	1,377	28	35
16B	18,050	13.06	1,382	20	24	83.4	83.4	1,247	27	32	1,455	13	18
16H	19,900	14.56	1,365	21	25	82.3	82.3	1,359	18	20	1,368	29	37
21A	20,180	14.84	1,360	22	29	82.0	82.0	1,360	17	19	1,355	30	39
23A	17,580	12.94	1,359	23	30	82.0	82.0	1,195	29	35	1,450	14	20
20A	19,740	14.66	1,347	24	31	81.2	81.2	1,329	21	24	1,355	31	39
22A	18,450	13.81	1,336	25	33	80.6	80.6	1,233	28	33	1,391	27	34
10A	19,375	14.5	1,336	26	33	80.6	80.6	1,294	23	26	1,357	30	38
16F	16,740	12.58	1,331	27	35	80.3	80.3	1,114	31	39	1,455	13	18
23B	15,600	11.73	1,330	28	36	80.2	80.2	1,037	34	45	1,506	6	10
10E	16,790	12.7	1,322	29	37	79.7	79.7	1,109	32	40	1,443	18	25
18A	15,320	12.1	1,266	30	39	76.4	76.4	970	35	46	1,446	15	21
1A	20,625	16.3	1,265	31	40	76.3	76.3	1,265	24	27	1,245	38	48
16E	14,960	11.97	1,250	32	41	75.4	75.4	936	37	48	1,444	18	24
1B	20,625	16.5	1,250	32	41	75.4	75.4	1,250	25	30	1,231	39	49
14D	19,690	15.81	1,245	34	43	75.1	75.1	1,225	26	31	1,255	37	47
10D	17,010	13.8	1,233	35	44	74.4	74.4	1,050	33	43	1,337	33	41
16D	14,850	12.07	1,230	36	45	74.2	74.2	914	40	51	1,428	25	31
31C	22,120	18.52	1,194	37	48	72.0	72.0	1,194	30	36	1,136	41	53
11A	15,800	13.4	1,182	38	49	71.3	71.3	938	36	47	1,328	34	42
10C	15,900	13.8	1,152	39	50	69.5	69.5	916	39	50	1,292	35	45
14A	15,500	13.8	1,123	40	51	67.7	67.7	870	41	53	1,275	36	46
40B	18,300	18.21	1,005	41	53	60.6	60.6	920	38	49	1,051	43	55
12A	15,875	15.9	999	42	54	60.3	60.3	794	42	54	1,121	42	54
13A	14,375	14.5	992	43	55	59.8	59.7	712	43	55	1,168	40	52
BEAM TESTED WITH ADDITIONAL LATERAL SUPPORT													
17A	16,170	10.4	1,555	1	6	100.0	93.8	1,257	5	29	1,730	2	2
30A	20,800	13.7	1,518	2	7	97.6	91.6	1,518	1	6	1,489	5	14
17B	15,300	10.1	1,515	3	8	97.4	91.4	1,159	8	37	1,732	1	1
30B	17,230	12.3	1,401	4	20	90.1	84.5	1,207	6	34	1,509	4	9
32B	19,660	14.40	1,365	5	25	87.8	82.3	1,342	3	22	1,377	8	35
14C	22,100	16.2	1,363	6	27	87.7	82.2	1,363	2	18	1,296	10	44
14B	18,500	13.6	1,361	7	28	87.5	82.1	1,259	4	28	1,415	7	32
23B-2	15,760	11.73	1,344	8	32	86.4	81.1	1,059	11	42	1,514	3	7
11B	16,250	12.4	1,310	9	38	84.2	79.0	1,065	10	41	1,454	6	19
32A	17,250	14.28	1,207	10	46	77.6	72.8	1,041	12	44	1,300	9	43
12B	19,240	16.0	1,202	11	47	77.3	72.5	1,156	9	38	1,225	11	50
13B	16,425	15.0	1,095	12	52	70.4	66.0	900	13	52	1,208	12	51
ECCENTRICALLY LOADED SPARS													
3F	20,720	10.77	1,925	1	------	100.0	---------	1,925	1	------	1,892	3	------
19B	18,440	9.98	1,848	2	------	96.0	---------	1,703	2	------	1,925	2	------
3D	18,680	10.64	1,756	3	------	91.2	---------	1,640	3	------	1,818	4	------
19A-2	14,580	8.6	1,695	4	------	88.1	---------	1,235	5	------	1,986	1	------
3C	18,110	10.7	1,693	5	------	87.9	---------	1,533	4	------	1,779	5	------
3A	15,400	10.0	1,540	6	------	80.0	---------	1,185	6	------	1,755	6	------
40A	15,270	11.1	1,376	7	------	71.5	---------	1,050	7	------	1,575	8	------
31B	16,700	12.4	1,348	8	------	70.0	---------	1,125	8	------	1,475	9	------
3B	14,070	10.5	1,340	9	------	69.6	---------	943	9	------	1,597	7	------
31A	14,125	12.7	1,112	10	------	57.8	---------	786	10	------	1,323	10	------

22. Table 4 gives the strength-weight ratios of the various spars tested and several related quantities. In Table 4 the spars are divided into three groups: Spars tested under design conditions, spars tested with additional lateral supports, and eccentrically loaded spars. This was done because the purpose of the division in carrying out this series of tests is to determine the relative merits of different types of construction, and it is considered that the variations in strength-weight ratio, as well as any other properties that might be used as figures of merit, caused by the use of additional lateral supports or by the devices of eccentric application of the axial load and camber, are too great to permit direct comparisons between spars of the different groups. If it had been thought possible to devise some correction factor that would permit direct comparison, this would have been done, but no such factor has yet been suggested.

23. In determining the standings in column 5 of Table 4 only the spars of the particular group are

considered, while for column 6, both of the first two groups are combined to show what the effect would be of neglecting the factor of additional lateral support. The only types that showed up particularly well when aided by the additional supports were types 17 and 30. None of the spars of these types would have shown up well if tested with a single support as they were lacking in lateral stiffness. In both cases, however, the two spars tested were designed simultaneously, and new spars designed to have adequate lateral stiffness might be made which would show up well if tested with a single support as called for in the specification. This is particularly true of type 17, both spars of which were very light and failed under a low load. If more material were added to the chord members and the chords widened to increase the lateral stiffness, it is probable that this type would show up very well. Type 30 does not lend itself so well to increasing the lateral stiffness, as the spars tested contained about as much material as some of those which showed up well when tested under design conditions, and if the channel chord members were made wider, some type of added stiffening would be needed to prevent local buckling or crinkling failure.

24. In columns 7 and 8 giving the strength-weight ratios in terms of the best strength-weight ratio developed, the same system was followed. The figures in column 7 were based upon the best figure in the group, and that in column 8 on the best figure in the first two groups combined. The spars in Group 3 were not rated against those of the first two groups, as the effect of the eccentric application of load was to reduce materially the loads to which they were subjected, particularly in the cases of types 3 and 19, which were the only types in this group which showed up well in the tests. As it would have been just as easy to have tested any of the other types with an eccentrically applied load, it was not considered fair to the spars in the first two groups to rate them with those in Group 3 without some allowance for the reduction of load involved in the eccentricities in the latter group, and as no fair way to make such a correction was devised, it was considered best not to make any such comparison.

25. Column 9 shows the strength-weight ratio of each spar multiplied by the percentage of the design load carried, except that when more than the design load was carried, the excess was neglected. This modification penalized those spars which did not carry the design load, and is therefore a combined measure of strength-weight ratio and the ability of the designer to predict the failing load of his structure, a very important point in practical design. Columns 10 and 11 give the relative standings of the spars based upon this criterion, column 10 giving the relative standings in the group, and column 11, the standings in the first two groups combined.

26. In penalizing for failure to meet the design load, there is danger of doing a type of design an injustice, particularly as the greater the load carried, the greater, in general, will be the strength-weight ratio, as the added material may be placed at the points of weakness, and thus improve the action of the spar as a whole. Thus a type of spar which failed under 15,000 pounds load with a given strength-weight ratio should show a better ratio if enough material is added to allow it to carry 20,000 pounds. The variation of strength-weight ration with change in load carried is unknown and undoubtedly depends largely upon the type of design and the cause of the failure under low load. If the failure under low load was due to a local weakness of small extent, the possible increase in strength-weight ratio is quite large. In order to obtain a measure of the possibilities of the different types, designs that carried more than the design load should be penalized and the figures for those which failed under lower loads should be increased. The first attempt to obtain a merit figure of this kind was by dividing the strength-weight ratio by the percentage of the design load carried. This figure would therefore be—

$$\text{Figure of merit} = \frac{\text{Ultimate load}}{\text{Weight of spar}} \times \frac{\text{Design load}}{\text{Ultimate load}}$$

$$= \frac{\text{Design load}}{\text{Weight of spar}}$$

Such a rating, therefore, would be merely a rating inversely proportional to the weights of the spars submitted and would not give the desired results.

27. The next attempt was made by modifying the procedure by dividing the strength-weight ratio by the square root of the percentage of the design load carried. The resulting figures and standings are given in columns 12 to 14 of Table 4. Studies of the resulting figures are interesting, but no general conclusions can be drawn from them, except that it is impracticable to arrive at a figure that will represent the complete possibilities of the different types of spars when some of the basic data are from tests of spars that failed locally and could be strengthened greatly with little addition of material, and others failed because the spar as a whole was loaded to its limit and the entire length of the spar would have to be strengthened to obtain a material increase in strength. Thus Spar 20A showed a true strength-weight ratio of 1,347, while the figure of merit under discussion indicated that the possibilities of this type were to obtain a ratio of 1,365. This spar failed, however, in the end fitting, which was cut off before weighing. Spar 20B was practically the same, except in the fitting; in fact it weighed 0.1 pound less, yet it developed a strength-weight ratio of 1,615 in test. The same figure of merit for 20B was 1,490, the reduction being due to the overload carried, the figure indicating that if the strength were decreased to the design load that the ratio would be thus decreased. Evidently this figure of merit is of no value in predicting the action of type 20, as the spread is so great between the figure 1,355 obtained from the spar failing under less than the design load and the 1,490 indicated by the spar which carried an overload. Similar studies of other groups also indicate that this figure of merit is of little value.

TABLE 5.—*Apparent areas*

CONTINUOUS WEBS

Spar 1	A 2	W_t 3	A_a 4	A/A_a 5	$\frac{P}{W_t}$ 6
10A	1.471	14.5	1.709	0.861	1,336
10B	1.471	14.6	1.720	.856	1,383
10C	1.409	13.8	1.626	.866	1,152
10D	1.409	13.8	1.626	.866	1,233
10E	1.135	12.7	1.496	.758	1,322
12A	1.700	15.9	1.874	.907	999
12B	1.700	16.0	1.885	.902	1,202
13A	1.600	14.5	1.709	.936	992
13B	1.600	15.0	1.768	.905	1,095
14A	1.254	13.8	1.626	.771	1,123
14B	1.214	13.6	1.603	.757	1,361
14C	1.516	16.2	1.909	.794	1,363
14D	1.477	15.8	1.864	.792	1,245
18A	1.319	12.1	1.426	.924	1,266
18B	1.434	13.6	1.603	.894	1,401
21A	1.589	14.84	1.749	.908	1,360
21B	1.496	14.25	1.680	.890	1,472
21C	1.496	14.50	1.709	.876	1,410
21D	1.496	14.62	1.723	.868	1,499
23A	1.240	12.94	1.525	.813	1,359
23B	1.061	11.73	1.382	.768	1,344
ECCENTRICALLY LOADED TRUSSED WEBS					
19A	0.772	8.6	1.014	0.762	1,695
19B	.782	9.98	1.176	.664	1,848
31A	.469	12.7	.534	.878	1,112
31B	.469	12.4	.521	.900	1,348
40A	.835	11.1	1.308	.638	1,376
TRUSSED WEBS					
11A	1.228	13.4	1.579	0.778	1,182
11B	1.228	12.4	1.461	.840	1,310
15A	1.160	13.6	1.603	.723	1,419
16A	1.261	12.7	1.496	.843	1,457
16B	1.247	13.06	1.540	.810	1,382
16C	1.350	13.42	1.581	.854	1,397
16D	1.173	12.07	1.422	.825	1,230
16E	1.166	11.97	1.411	.826	1,250
16F	1.188	12.58	1.482	.801	1,331
16G	1.355	13.46	1.586	.854	1,497
16H	1.386	14.56	1.716	.808	1,365
17A	1.000	10.4	1.225	.816	1,555
17B	1.000	10.1	1.190	.840	1,515
20A	1.420	14.66	1.728	.822	1,347
20B	1.350	14.56	1.716	.786	1,615
22A	1.127	13.81	1.627	.692	1,336
22B	1.219	13.57	1.599	.763	1,459
30A	.416	13.7	.576	.722	1,518
30B	.416	12.3	.517	.804	1,401
31C	.657	18.52	.778	.844	1,194
32A	.459	14.28	.600	.765	1,207
32B	.459	14.40	.605	.759	1,365
40B	1.497	18.21	2.146	.698	1,005

STUDY OF APPARENT AREAS

28. Study of the test results confirms the impression that in order to design a spar with a high strength-weight ratio, the amount of material that does not aid in carrying the direct axial load must be kept to a minimum. The fact that the webs of box types are able to carry their share of the axial load seems to give them a slight advantage over trusses, the web members of which can not carry any of that load. In the size of spars tested, this effect was small, and if the spars had been deeper the other advantages of trussing would counteract it entirely. On the other hand, in shallower spars, the advantage of the boxes should be increased.

29. A measure of the merits of the different designs in this respect is furnished by the ratio of the cross-sectional area of the spar to its apparent area, this quantity being the weight divided by the density and the length of the spar. The computations of the apparent area and the ratio mentioned are given in Table 5. This table gives the sectional area A taken from Table 3, the apparent area, A_a, and their ratio. Figure 14 shows this ratio plotted against strength-weight ratio.

30. As the web area of trussed spars was assumed zero in Table 3, it is to be expected that a truss with a given ratio of A to A_a would have a better strength-weight ratio than a box with the same ratio. The points representing trusses are plotted with circles, while spars with continuous sheet webs are represented by crosses. Though the plotted points are quite scattered, they tend to fall about as expected, and

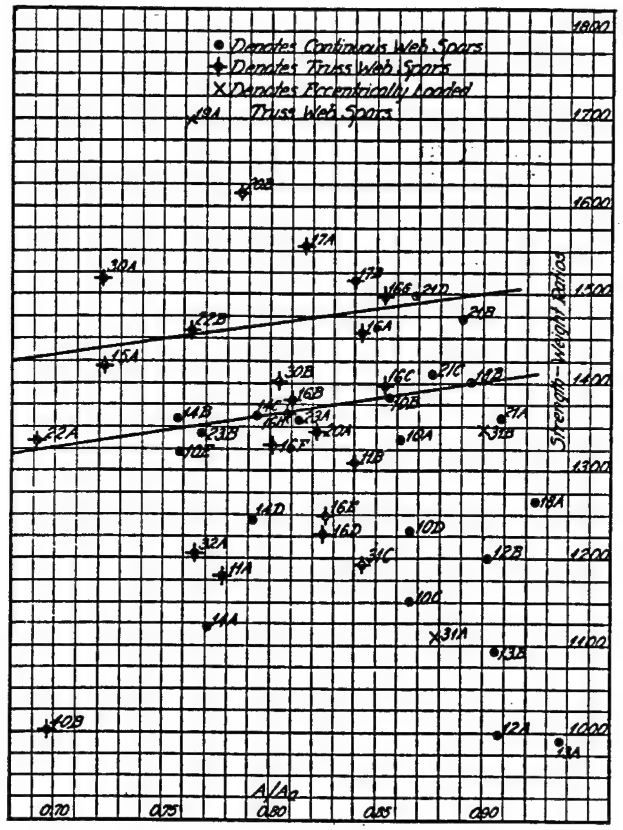

FIG. 14

curves can be drawn showing the envelope of each group. The fact that some spars did not show nearly as large a strength-weight ratio as is indicated for them by their ratio of A to A_a shows only that they were lacking in other qualities necessary for efficient design such as strength against lateral buckling of the compression chord.

STIFFNESS OF SPARS

31. One of the most important properties of a spar is its stiffness, a quantity that must be known if its strength is to be predicted with precision. The designer of a beam can compute the stiffness modulus EI, of his design by multiplying the standard value of Young's modulus, E, of the material used by the

moment of inertia, I, of the section, and the resulting value will usually give correct values for the deflection under load when used with the standard deflection formulas. If, however, there is slippage in the joints connecting the various members forming the beam, or there is appreciable shear deflection in the web, the EI value so obtained will not give correct results when used in the deflection formulas. The error will be specially pronounced when a truss type of spar is used, as the sizes of the web members used are not considered in the computations of the geometrical I of the cross section. If the deflections of a given spar are measured, however, it is a simple matter to compute the effective EI from them with the help of the beam formulas.

32. In the study under discussion, three effective EI values for all the metal spars tested were obtained by two methods. Before the main test under combined axial and side loads, each spar was tested in simple bending, the loads being kept below those which might cause permanent deformation, and the deflections measured for the determination of EI. This was done first with the loads perpendicular to the plane of the spar to obtain the value of effective EI about its major axis and then with the loads in the plane of the spar to obtain the effective EI about its minor axis. After the main test, in which the spar was tested to failure, the effective EI was again determined about the minor axis from the deflection readings of that test.

33. The various EI values and the ratios between them are given in Table 6. The nomenclature in this table and the following discussion is as follows:

- E_sI_{gx} Standard E times geometric I about minor axis.
- EI_{xx} Effective EI about minor axis from cross-bending test.
- EI_{mt} Effective EI about minor axis from main test on spar.
- E_sI_{gy} Standard E times geometric I about major axis.
- EI_{yy} Effective EI about major axis from cross-bending test.

34. Column 11 of Table 6 gives the ratio of EI_{xx} to the weight of a 7-foot section of spar and column 12 the ratio of EI_{mt} to the same weight.

TABLE 6.—*Stiffness properties of spars*

Spar	E_sI_{gx} [1]	EI_{xx}	EI_{mt}	$\frac{EI_{xx}}{E_sI_{gx}}$	$\frac{EI_{mt}}{E_sI_{gx}}$	$\frac{EI_{mt}}{EI_{xx}}$	E_sI_{gy} [1]	EI_{yy}	$\frac{EI_{yy}}{E_sI_{gy}}$	$\frac{EI_{xx}}{W_t}$	$\frac{EI_{mt}}{W_t}$
1	2	3	4	5	6	7	8	9	10	11	12
10A	91.8	75.2	88.0	−0.820	+0.959	+1.170	11.97	8.18	−0.684	5.18	6.07
10B	91.8	91.2	97.0	+.994	+1.056	+1.064	11.97	11.97	−1.000	6.24	6.64
10C	94.5	86.6	95.3	+.916	+1.009	+1.100	11.26	10.97	−.974	6.28	6.91
10D	94.5	88.2	90.3	+.934	+.956	−1.024	11.26	11.71	+1.040	6.39	6.54
10E	92.7	90.6	80.4	+.978	−.867	−.888	9.05	9.31	+1.029	7.13	6.33
11A	68.4	[2] 53.4	51.6	−.781	−.754	−.965	5.32	4.88	−.918	3.99	3.85
11B	68.4	54.6	52.8	−.798	−.772	−.968	5.32	5.34	−1.004	4.40	4.26
12A	87.6	89.0	85.9	+1.016	+.981	−.965	4.50	4.39	−.976	5.60	5.40
12B	87.6	83.5	85.7	+.953	+.978	−1.026	4.50	4.50	−1.000	5.22	5.36
13A	82.0	72.8	73.0	−.888	−.890	−1.003	4.06	4.10	−1.010	5.02	5.04
13B	82.0	76.0	74.5	+.927	−.908	−.980	4.06	3.88	−.956	5.07	4.96
14A	93.4	78.3	98.8	−.838	+1.057	+1.262	4.23	4.76	+1.125	5.68	7.16
14B	91.0	96.0	92.7	+1.055	+1.019	−.965	3.96	4.76	+1.201	7.06	6.82
14C	112.9	106.7	102.0	+.945	−.904	−.956	4.63	5.87	+1.267	6.58	6.30
14D	112.0	133.5	109.0	+1.192	+.973	−.816	6.05	6.86	+1.133	8.44	6.89
15A	92.1	81.1	83.4	−.881	−.905	+1.028	8.54	8.46	−.991	5.96	6.13
16A	99.3	77.5	85.0	−.781	−.856	+1.096	16.57	16.92	+1.021	6.10	6.69
16B	102.7	83.7	107.2	−.815	+1.044	+1.280	18.34	18.48	−1.008	6.40	8.20
16C	109.0	87.1	102.8	−.799	+.943	+1.180	20.39	20.70	−1.015	6.49	7.66
16D	87.6	79.9	86.2	+.912	+.984	+1.079	13.40	13.13	−.980	6.62	7.14
16E	82.3	72.3	86.6	−.879	+1.052	+1.197	13.35	13.62	+1.020	6.04	7.23
16F	102.1	78.0	87.9	−.764	−.860	+1.127	17.93	18.35	+1.024	6.20	6.98
16G	107.7	84.6	103.6	−.785	+.962	+1.224	20.11	20.20	−1.005	6.28	7.69
16H	110.3	87.4	99.1	−.792	−.898	+1.134	21.00	19.53	−.930	6.00	6.80
17A	67.7	60.7	61.2	+.897	−.904	−1.009	5.42	5.53	+1.020	5.83	5.88
17B	63.2	61.3	57.8	+.970	−.915	−.943	5.33	5.49	+1.030	6.07	5.72
18A	70.7	69.8	64.8	+.988	−.916	−.928	5.33	4.74	−.890	5.77	5.36
18B	79.2	80.1	78.3	+1.011	+.989	−.978	9.82	9.14	−.931	5.89	5.76
19A	45.4	33.4	--------	−.736	--------	--------	11.40	8.26	−.724	3.88	--------
19B	38.3	31.4	--------	−.820	--------	--------	15.68	13.65	−.871	3.15	--------
20A	103.3	93.9	83.7	+.909	−.810	−.892	8.66	9.[illegible]	+1.075	6.42	5.70
20B	100.8	91.4	86.0	+.907	−.853	−.941	8.46	8.75	+1.034	6.28	5.90
21A	113.7	93.4	92.5	−.822	−.813	−.990	10.34	9.50	−.919	6.30	6.23
21B	102.1	87.4	100.8	−.856	+.987	+1.153	10.21	9.55	−.935	6.14	7.08
21C	102.1	100.1	101.7	+.980	+.996	−1.015	10.21	10.13	−.993	6.90	7.01
21D	102.1	97.5	99.6	+.955	+.975	−1.021	10.21	10.55	+1.033	6.67	6.81
22A	90.1	74.4	69.9	−.826	−.776	−.940	12.19	13.27	+1.089	5.38	5.06
22A	100.1	82.9	75.1	−.828	−.750	−.906	13.14	13.08	−.995	6.11	5.53
23A	59.3	68.1	73.0	+1.148	+1.230	+1.072	5.82	7.09	+1.217	5.26	5.64
23B	56.0	62.7	68.5	+1.109	+1.223	+1.092	5.78	6.71	+1.160	5.34	5.83
30A	102.4	89.0	86.0	−.869	−.840	−.966	5.61	5.78	+1.030	6.50	6.28
30B	102.4	93.1	83.9	+.909	−.819	−.901	5.61	5.82	+1.037	7.57	6.82
31A	93.7	72.4	--------	−.773	--------	--------	5.34	5.28	−.988	5.70	--------
31B	89.4	74.1	--------	−.829	--------	--------	5.34	5.38	−1.007	5.98	--------
31C	119.3	97.1	103.8	−.812	−.869	+1.069	11.20	12.09	+1.079	5.24	5.60
32A	89.3	89.0	76.0	+.997	−.851	−.854	5.54	6.09	+1.100	6.23	5.32
32B	86.3	87.0	74.5	+1.008	−.863	−.856	5.54	6.39	+1.153	6.04	5.17
40A	56.0	45.5	--------	−.812	--------	--------	4.37	5.01	+1.144	4.10	--------
40B	84.7	54.3	66.3	−.641	−.782	+1.220	18.12	18.70	+1.031	2.98	3.64
Average values				0.895	0.9261	1.028			1.016		
Average error				.0869	.0841	.0926			.0694		

[1] E for dural sheet 10.0, dural tube and extruded section 10.4, steel 29.0.
[2] EI by proportion from spar 11B.

35. The unit used in Table 6 for the various EI values is 1,000,000 pound-inch units. In the columns giving the ratios of the EI values to each other, the decimal point is in the natural location. In columns 11 and 12 the figure shown is the ratio of EI in millions of pound-inch units to the weight in pounds.

36. The geometrical EI values about both axes of the cross section were obtained by multiplying the values of I given in Table 3 by the standard values of Young's modulus for the materials in the chord members. For this purpose the following values of E were used: Duralumin sheet, 10,000,000 pounds per square inch; extruded duralumin, 10,400,000 pounds per square inch; and steel, 29,000,000 pounds per square inch.

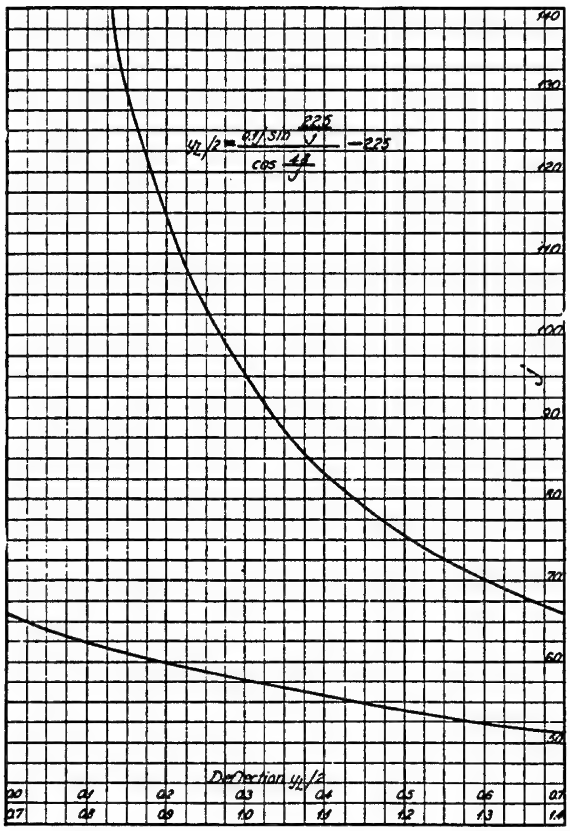

FIG. 15

37. The effective EI values from the tests in simple bending were obtained by measuring the deflections, substituting the values obtained in the ordinary formulas for the deflection of beams and solving for EI. The procedure in the case of the EI values obtained from the tests under combined axial and bending loads was not so simple. The formulas for moments and deflections under loading of the type used in the tests are given in Serial Report 2744 "Precise Formulas for Restrained Beams with Axial Load of Compression, Equal End Moments, and Two Symmetrically Placed Side Loads," by A. S. Niles. The formula for the deflection in the center of the span as given in that report is—

$$y_{L/2}=-\frac{1}{P}\left[\frac{M_1-Wj\sin\frac{a}{j}}{\cos\frac{L}{2j}}-W_a-M_1\right]$$

When M_1 is zero, L is 96 inches, a is 22.5 inches, and W is 0.1P, this formula reduces to

$$y_{L/2}=\frac{0.1j\sin\frac{22.5}{j}}{\cos\frac{48}{j}}-2.25$$

Where y is the deflection and $j=\sqrt{\frac{EI}{P}}$

38. As EI appears in this formula as part of a complex trigonometric expression it is not possible to solve directly for it when all of the other quantities are known. In order to overcome this difficulty, the formula for the special case being considered—in which y appears as a function of j—was plotted as shown in Figure 15. The use of this curve made it possible to determine the value of j corresponding to any measured deflection, and from there the computation of EI was simple. No attempt was made to compute the effective EI values of the spars with camber or eccentric application of the axial load.

39. The values of EI_{mt} were first computed from the deflection corresponding to an axial load of about 5,000 pounds, as it was feared that if the deflection under a larger load were used the EI value obtained would be too low, as the observed deflection might include the effects of some permanent deformation. This did not prove satisfactory, however, as theoretical deflections under axial loads of from 8,000 to 14,000 pounds computed from the EI_{mt} values obtained in this way were often greater than the observed deflections. This result did not seem reasonable and it was finally decided to accept as the EI_{mt} for each spar the largest value justified by its deflection under an axial load of 5,000 pounds or more.

40. In computing the EI_{mt} values from the observed deflections it was necessary to know the zero error of each load-deflection curve. To obtain this value, which was also needed to correct the observed deflection at maximum load in computing unit stresses at failure, the lower portion of the curve was extrapolated as a flat curve from the central portion. Owing to the difficulty of doing this with any certainty that the location and curvature of the extrapolated portion of the curve was chosen correctly, the extent of the zero errors could not be determined with precision. In some cases all of the observed deflections lay so close to a well defined curve of the proper shape that the error in EI_{mt} as well as in deflection at maximum load is very small. In others it may be quite appreciable as it was difficult to decide whether to give more weight to deflections observed under axial loads from about 2,000 to about 6,000 pounds, or those under loads of from about 8,000 to about 14,000 or 16,000 pounds. Unfortunately, the spars in which the reader is probably most interested, like 20B, were among those of the second class.

41. This source of error is of little importance in connection with the computations of maximum unit stresses, but was of considerable importance in the EI_{mt} computations. Owing to it, it is not possible to give as much weight as would be desirable to small differences in EI_{mt} in comparing spars, particularly those cases where the proper prolongation to zero of the load-deflection curves was most uncertain. The error involved, however, was not so great as to preclude certain generalizations from being made with propriety.

42. Seven of the 49 spars tested showed values of EI_{xx} greater than E_sI_{gx}, but the average value of the former quantity was 10.7 per cent less than the latter. The average deviation from the mean was a little less than 9 per cent. In other words, on the average, the effective EI of the spars tested was only 90 per cent (more precisely 89.3 per cent) of the EI to be obtained as the product of the standard E of the material used and the moment of inertia of the section. In any given case, however, the probability would be that the ratio of effective to geometric EI would vary from the mean by about 9 per cent. This variation in the ratio of effective to geometric EI was practically constant for several types, but was quite erratic with others.

43. From theoretical grounds it would seem that on account of the shear deformation of continuous webs, and the deformation of the web members of trusses, that none of the spars could show a ratio of effective to geometrical EI greater than unity, that the extruded I sections would show ratios a little below unity, followed rather closely by the plate girders and boxes with continuous sheet webs, and at some distance by the trusses. On the whole this order is followed, but with some rather striking exceptions. One cause that might operate in the cases of any of the types is the possibility that the material actually used possessed a higher Young's modulus than the standard value attributed to it. A similar possibility, particularly where the cross section is made up of corrugations or other irregular sections, is that the geometrical I computed for the section is incorrect. The spars with ratios over 1 and the corresponding ratios are the following: 12A, 1.016; 14B, 1.055; 14D, 1.192; 18B, 1.011; 23A, 1.149; 23B, 1.119; and 32B, 1.008.

44. 12A is an extruded I beam so it should have a high ratio. The value of E of its web material is slightly lower (10.32) than the standard value, but the curvature of the chords, the bulbs at the free edges of the chords, and the fillets at the junctions of the chords with the web made it difficult to compute the true value of I. On the other hand, spar 12B which was extruded through the same die, and for which the same value of I was used, showed a value of 10.5 for E, yet the observed EI_{xx} was only 95 per cent of the computed value of E_sI_{gx}. The difference is hard to explain, unless the samples of web from which the values of E shown in Table 2 were obtained were not truly representative or the precision of the tests for EI_{xx} much less than is believed to be the case, though the necessity of working with low loads makes that precision less than is desirable.

45. Two spars of type 14 show values of EI_{xx} greater than their values of E_sI_{gx}, 14B and 14D. In the case of 14B, the E of the chords from Table 2 is sufficiently greater than the standard value to explain most of the difference, but not great enough for the case of 14D. In fact no satisfactory hypothesis has been developed to account for the range of the ratio of EI_{xx} to E_sI_{gx} from 0.838 for 14A to 1.192 for 14D.

46. Both spars of type 18 showed high ratios, that for 18A being slightly under and that for 18B slightly over unity. As the value of I for 18B was determined by proportion from that of 18A, and the latter was obtained graphically, the high ratios may be due to errors in computing I. Table 2 also indicates that the material used had considerably higher values of Young's modulus than the standard value assumed.

47. Both spars of type 23 showed very high ratios. In their cases, the values of I were obtained graphically from a figure that was an attempt at reconstructing the original section after it had been injured in the test. It seems most reasonable to believe that the computed value of I_{gx} for these spars was too low.

48. The spars of type 32 showed very high ratios in spite of the fact that they were trusses. In this case, the most reasonable explanation seems to be that the E of the steel used was nearer 30,000,000 than 29,000,000 and that the welded joints and the method of joining the tension web members to the chords were responsible for this result.

49. It is obvious that the effective EI value obtained for any given design will depend to some extent on the character of the loading used in obtaining the deflections from which EI is computed. It is only reasonable, therefore, to expect that the values of EI_{mt} will differ from the values of EI_{xx}, but it might reasonably be expected that the ratio of EI_{mt} to EI_{xx} would be approximately constant. The actual values of these effective EI quantities do differ, and it is of interest that the average ratio of EI_{mt} to E_sI_{gx} is somewhat greater than the average ratio of EI_{xx} to E_sI_{gx}, the figures being 0.926 and 0.893, respectively. The average deviation from the mean for the ratio EI_{mt} to E_sI_{gx} is slightly smaller than the corresponding quantity for the other ratio, being 0.0841 against 0.0874. Generally speaking, the two ratios for the various spars change in about the same manner, but the correlation is not very close. Eight spars show ratios of EI_{mt} to E_sI_{gx} greater than unity, but only three of these, 14B, 23A, and 23B, are spars that have such ratios of EI_{xx} to E_sI_{gx}. The eight spars with their ratios are: 10B, 1.056; 10C, 1.009; 14A, 1.057; 14B, 1.019; 16B, 1.044; 16E, 1.052; 23A, 1.230; and 23B, 1.223.

50. 10B and 10C are boxes, and in the case of 10B at least, the E of the material is considerably higher than the standard value. 14A and 14B are plate girders. Interestingly enough, 14A showed the lowest ratio of EI_{xx} to E_sI_{gx} of the four plate girders, but is the highest with respect to EI_{mt}. 16B and 16E are duralumin channel trusses and should not show particularly high ratios, but they do nevertheless.

51. The writer has been unable to obtain any satisfactory hypothesis for the relations between the EI_{xx} and EI_{mt} values. Table 6 shows the ratio of the latter to the former for each of the spars. The average value of this ratio is 1.028 and the average deviation from the mean, 0.0924. In most cases the effective EI obtained

from simple bending is less than that obtained from the test under combined load.

52. In most of the spars the agreement between E_sI_{gy} and EI_{yy} was very close, more than half of the values in column 10 of Table 6 falling between 0.95 and 1.05, indicating that in most cases the lateral stiffness of the spars can be computed from the sectional dimensions with an error of less than 5 per cent. When the data of this column is plotted in the form of a frequency curve, they show a well defined mode, for the number of tests available, between 1.02 and 1.03. The value of 1.02 is in very close agreement with the arithmetic mean value of 1.017, though this is largely accidental.

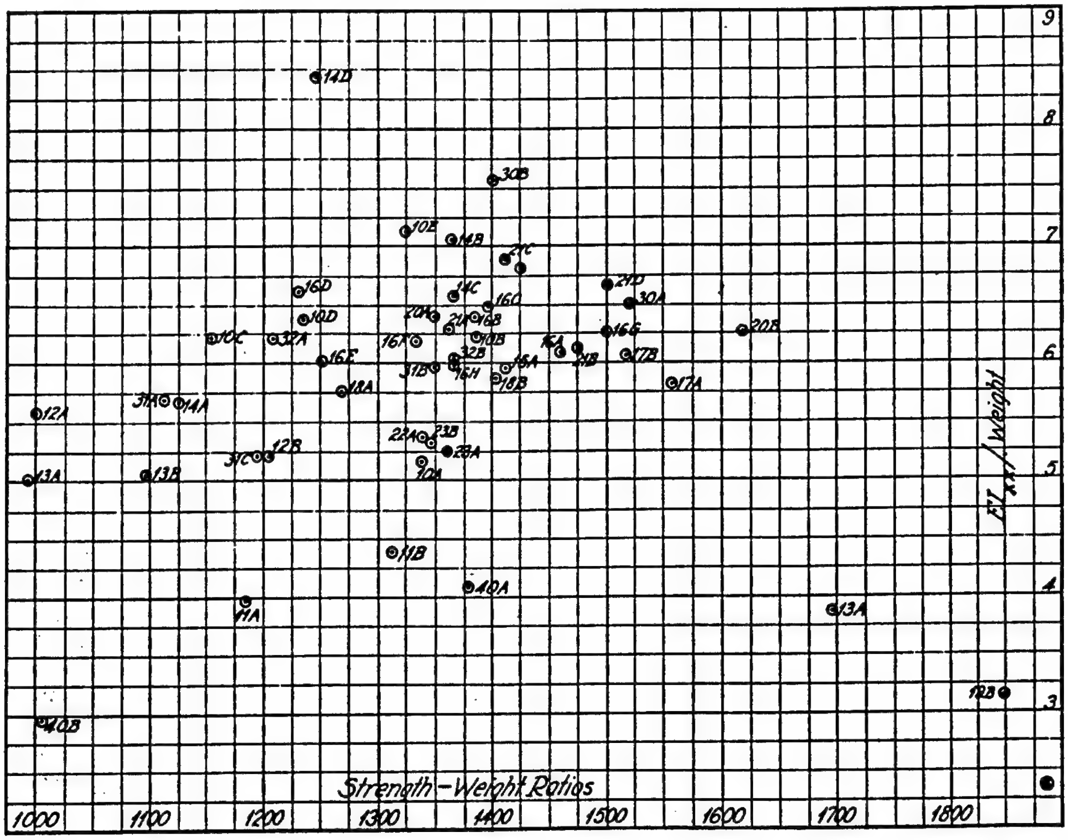

FIG. 16

53. Some of the types showed a well defined tendency to have lower values of the ratio of EI_{yy} to E_sI_{gy} than 0.95, and other types showed a similar tendency to show higher values than 1.05. Both spars of type 18 and both of type 19 showed low ratios. In the case of type 18 this may have been due to errors in computing the moment of inertia of the irregular sections used. In the case of type 19 it was almost certainly due to the light connecting members between the two main tubes of the compression chord. Two of the spars of type 21 and one of type 10 also showed low ratios, but these are balanced by the normal ratios shown by the other spars of these types. All four spars of type 14 showed high ratios, due probably to higher than normal values for E of the material actually used and to errors in computing the geometric I due to the bulbs at the corners of the chord members. Type 23 also showed high ratios, due probably to errors in computing the geometric I. These spars, it may be remembered, also showed ratios of EI_{xx} and EI_{mt} to E_sI_{gx} greater than unity. The reason for the high values shown by the spars of types 32 and 40 is not known unless it was due to the neglect of webs in computing the value of I or error involved in assuming the chord members to be true ellipses.

54. The assumed values for E_s were conservative, and this probably accounts for the modal value of the ratio being slightly above 1. If the actual values of E had been known and used instead of E_s the probability is that the modal value would have been slightly below 1. That the modal value is so near to 1 is due to the fact that the geometric I about the major axis of the spar is made up almost entirely of the moments of inertia of integral chord members like channels, the cover plates of boxes, or elliptical tubes, and the lateral stiffness of these members is not affected by any joints. The only type in which the compression chord is made up of two members connected by what could be considered a light web is type 19, and as remarked above, both of these spars showed very low ratios of EI_{yy} to E_sI_{gy}.

55. Just what the factors are which determine the relationship between the various values of effective EI would make an interesting study, but the data available at present is not sufficient to warrant going further into this subject in this report.

56. The values of effective EI divided by the weight in the last two columns of Table 6 are measures of the efficiency of the location of the material in the spars in regard to obtaining stiffness. One would expect that the more efficiently the material was located for the production of stiffness, the less the deflections and secondary bending moments would be and the greater the strength-weight ratio. The figures of EI divided by weight are plotted against strength-weight ratio in Figures 16 and 17. In Figure 16 EI_{xx} is used, and the figure seems to indicate that there is a tendency, though none too well defined, to obtain the result mentioned above. At the point where the proportional limit of the material is reached, the slope of the curve decreases, the curve thus diverging from the straight line. Below the proportional limit, if the load is removed, the deformation will return to zero. If the proportional limit has been passed before the load is removed, the deformation will not return to zero, but the test piece will show a residual deformation called the permanent set. That part of the deformation which disappears when the load is removed may be called the elastic deformation, while the part which will not disappear because the material has been permanently changed in shape, may be called the plastic deforma-

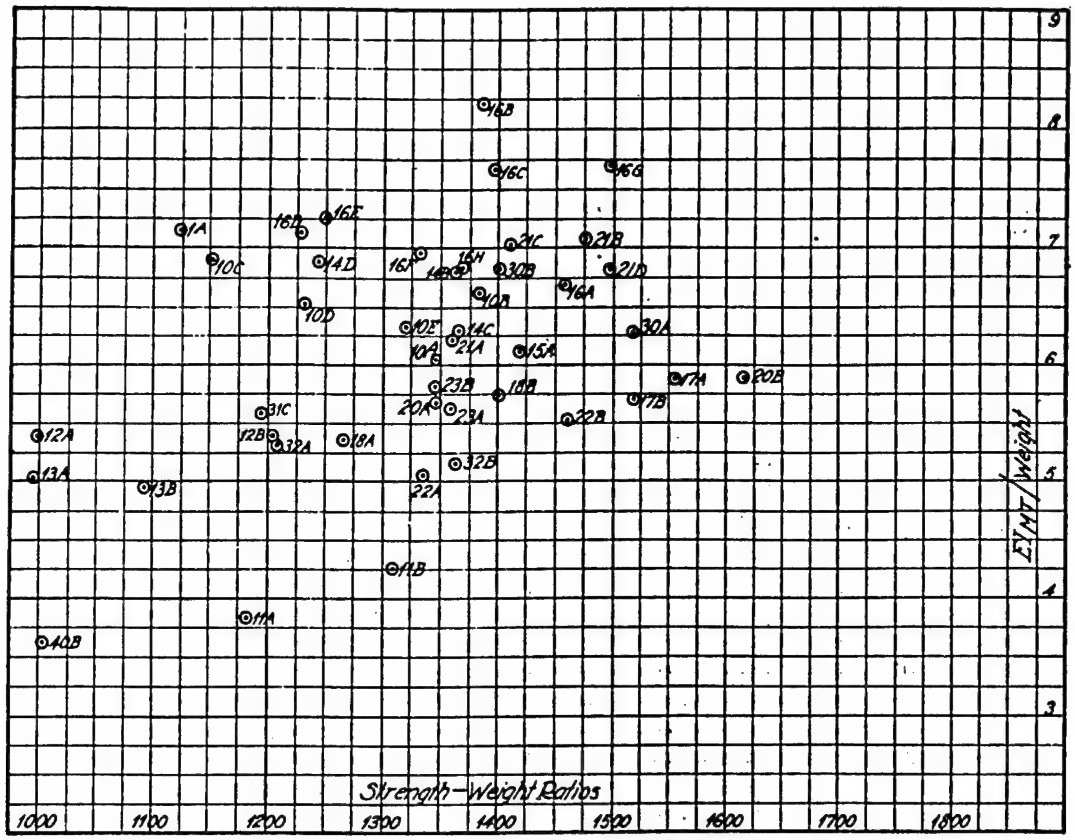

FIG. 17

However, close study of the figure will show that when the spars of each type are connected together the rule fails to hold for more types than these to which it applies. In Figure 17, which is based on EI_{mt}, the points are more scattered, and one could use the figure to prove the opposite result as well as the one expected. The magnitude of the possible differences in secondary bending moments is evidently too small in comparison with the total bending for the efficiency of location of material to provide stiffness to have much effect on the strength-weight ratio.

PLASTIC DEFORMATION

57. If a load-deformation curve is plotted for a simple beam subjected to side or bending loads only, the lower part of the curve will be a straight line. tion. In practice, the elastic deformation under a given load is the abscissa to the straight-line portion of the load-deformation curve, extended, if necessary, at the ordinate corresponding to the load in question. The remainder of the abscissa to the actual load-deformation curve will represent the plastic deformation.

58. In the case of beams subjected to combined axial and bending loads, the division of deflection between elastic and plastic deformation can not be obtained in just this manner, though the underlying theory is identical. For this case, the straight-line portion of the load-deformation curve is replaced by a load-deflection curve obtained by plotting the precise formula used in computing EI_{mt} as a load-deflection curve. To do this, a value of EI of the beam is needed, and EI_{mt} is con-

sidered the most appropriate one available for the purpose. After the curve of elastic deflection has been plotted, the differences in the deflections shown by it and those observed can be considered the plastic deflection.

59. If the actual load-deflection curves for the spars tested are plotted beside the theoretical load-deflection curves obtained from the use of the formula for the loading used and EI_{mt} it will be seen that the spars fall into two groups between which there is no sharp dividing line. In the first group the curve of observed deflections remains close to the theoretical curve until the maximum load is reached, when it stops short, indicating a sudden failure of the spar. In the second group, the two curves diverge at a lower load, and are quite far apart when the maximum load is reached. In some cases, the curve of observed deflections rises to a maximum and then drops, collapse of the spar occurring at a lower load than the maximum.

60. Spars with load-deflection curves of the first type may be said to have failed suddenly and to be lacking in toughness, while spars with the second type of curve may be said to fail gradually and to possess toughness. It is realized that the use of the word toughness in this sense is open to criticism, but it is expressive of the impressions produced by the two types of spar and will be used in this report with the meaning indicated above. As is only to be expected from the varied natures of the spars tested, there is no sharp dividing line between those which lack and those which possess toughness.

61. Table 7 gives data from which a study of the relative toughness of the spars tested can be made. This table gives the computed and observed deflections under maximum load, 0.8 maximum load, and 0.6 maximum load for the 44 metal spars tested under design conditions, and also those tested with additional lateral support. The computed deflections were based upon the value of EI_{mt} given in Table 6. The ratio of the observed deflection to the computed deflection is given in each case. The value of this ratio minus unity may be considered to represent the ratio of the plastic deformation to the elastic, and is nearly always positive. In the few cases where it is negative, the value is less than 1 per cent, which is within the precision of the computations, and in all such cases, the result may be interpreted to mean that the plastic deformation was either zero or too small to make its presence evident.

TABLE 7.—*Study of plastic deflection*

Spar	At maximum load			At 0.8 maximum load			At 0.6 maximum load		
	δ_o	δ_c	δ_o/δ_c	δ_o	δ_c	δ_o/δ_c	δ_o	δ_c	δ_o/δ_c
1	2	3	4	5	6	7	8	9	10
10A	0.701	0.669	1.048	0.510	0.507	1.005	0.363	0.363	1.000
10B	.950	.624	1.523	.545	.474	1.149	.358	.341	1.050
10C	.700	.476	1.470	.412	.368	1.120	.278	.267	1.041
10D	.661	.550	1.201	.431	.423	1.019	.304	.306	.994
10E	.760	.626	1.214	.490	.476	1.030	.344	.343	1.002
11A	1.450	1.032	1.405	.810	.762	1.063	.550	.537	1.024
11B	1.670	1.040	1.606	.807	.765	1.055	.537	.538	.998
12A	.630	.538	1.170	.420	.413	1.016	.299	.299	1.000
12B	.800	.692	1.156	.532	.525	1.013	.374	.371	1.008
13A	.611	.581	1.051	.445	.443	1.004	.319	.320	.997
13B	.800	.673	1.189	.515	.507	1.015	.363	.365	.995
14A	.510	.442	1.154	.356	.344	1.035	.252	.250	1.008
14B	.719	.593	1.212	.457	.450	1.015	.326	.326	1.000
14C	.800	.657	1.217	.500	.497	1.006	.363	.358	1.014
14D	.650	.522	1.245	.408	.402	1.015	.292	.292	1.000
15A	.985	.713	1.381	.548	.538	1.018	.382	.385	.992
16A	.960	.661	1.452	.510	.499	1.021	.359	.359	1.000
16B	1.000	.479	2.089	.405	.370	1.095	.270	.270	1.000
16C	1.085	.531	2.042	.431	.408	1.056	.295	.295	1.000
16D	1.150	.493	2.331	.403	.381	1.057	.279	.277	1.007
16E	1.280	.495	2.587	.418	.382	1.094	.282	.278	1.015
16F	.665	.561	1.185	.505	.430	1.174	.365	.310	1.177
16G	1.180	.571	2.066	.477	.438	1.089	.325	.316	1.029
16H	1.073	.598	1.795	.487	.455	1.070	.333	.328	1.015
17A	1.014	.838	1.210	.645	.635	1.015	.450	.447	1.006
17B	1.000	.843	1.185	.638	.638	1.000	.454	.449	1.011
18A	.950	.730	1.301	.572	.554	1.032	.395	.396	.998
18B	1.015	.753	1.348	.622	.574	1.083	.412	.407	1.012
20A	1.172	.727	1.611	.556	.550	1.010	.394	.394	1.000
20B	1.195	.882	1.355	.675	.663	1.018	.467	.466	1.002
21A	.860	.662	1.299	.499	.501	.996	.365	.361	1.011
21B	1.290	.627	2.058	.633	.475	1.333	.423	.341	1.240
21C	.845	.600	1.408	.486	.455	1.068	.333	.330	1.010
21D	1.005	.670	1.500	.560	.505	1.109	.370	.363	1.020
22A	.900	.838	1.074	.633	.635	.997	.455	.447	1.018
22B	1.750	.833	2.100	,859	.634	1.355	.450	.446	1.010
23A	1.010	.746	1.354	.587	.566	1.037	.405	.403	1.005
23B	.900	.709	1.270	.539	.536	1.005	.381	.382	.998
30A	.936	.751	1.246	.590	.569	1.037	.416	.406	1.025
30B	.713	.617	1.155	.481	.468	1.028	.339	.338	1.003
31C	.880	.643	1.369	.492	.485	1.015	.350	.350	1.000
32A	1.070	.695	1.540	.548	.525	1.044	.377	.377	1.000
32B	1.110	.838	1.325	.662	.635	1.042	.450	.446	1.010
40B	1.813	.893	2.030	.815	.673	1.211	.491	.472	1.040
Total			64.527			46.669			44.785
Average			1.4665			1.0606			1.0178
Average excluding 16F and 21B			1.4591			1.0514			1.0087
Mode						1.01–1.02			1.00–1.01
Median			1.351			1.033			1.01
Median first 3d			1.217			1.016			1.00
Median second 3d			1.470			1.057			1.01

δ_o=observed deflection. δ_c=computed deflection.

The observed deflections listed in the second column of Table 7 are in some cases different from those given in Table 1; the former are the corrected deflections, obtained by correcting the deflection curve for zero error. This was done by prolonging the lower portions of the deflection curve down to the axis of abscissas, and then shifting the entire curve so that it passed through the origin. In this way, an attempt was made to correct the zero error due to possible slackness of the cord from the spar to the deflectometer, and the fact that the dial was set at zero when the true axial load on the spar was not zero, but from 100 to 312.5 pounds. The precision of this correction is none too good in many cases, as it was difficult to determine the proper shape for the lower part of the load-deflection curves. A further error was also introduced in some cases by the fact that the spar failed too suddenly to permit the deflection at maximum load to be read or the load deflection curve was so flat that it was difficult to determine the deflection at which maximum load was reached.

62. Study of Table 7 brings out some very interesting facts. At 0.6 maximum load, only two spars, 16F and 21B show plastic deflections greater than 5 per cent of the elastic deflections, and in both of these cases the figures are open to question. The EI_{mt} value used for spar 16F was derived from the first test, while the observed deflections were taken from the second test. If the EI used had been the higher value taken from the second test, the plastic deformation would have been as small as in the other cases. In the case of spar 21B, the EI value used was derived from the deflection under a rather low load, and it is thought that it is unduly high, though this can not be proved. If these two spars be omitted from consideration, the average plastic deformation of the remaining 42 spars is only 0.87 of 1 per cent of the elastic deflection, and even if they are included, the average is raised to only 1.78 per cent. Thirty-seven of the 44 spars showed less than 3 per cent plastic deflection. It may be safely said, then, that at 0.6 maximum load, the plastic deflection is in general negligible.

63. At 0.8 maximum load the plastic deflection has increased, the average being 6.06 per cent of the elastic deflection if all spars are included, and only 5.14 per cent if the two doubtful cases be neglected. On the whole, however, the plastic deflection is still low, it being 3.3 per cent or less in half of the spars tested. The minimum value is minus 0.4 per cent which may be considered as zero and the maximum 35.5 per cent for spar 22B. The mean value is considerably higher than the median as there were several spars with considerable amounts of plastic deflection, while those which had none could not bring down the average very far. At this load it may be said that while the average plastic deflection is still small, and in many cases zero or negligible, in several cases it is quite appreciable.

64. At maximum load the plastic deflection ranges from a minimum of 4.8 per cent for spar 10A to 158.7 per cent for spar 16E. The mean value is about 46 per cent and the median 35 per cent. These figures show the great differences in plastic deformation that were met with and illustrate the difference in action between the spars which lacked and those which possessed toughness. On the whole, the spars with single thin webs like types 12, 13, 14, and 23 showed themselves to be lacking in toughness, while boxes and channel trusses, like types 10, 16, 21, and 22, usually possessed this quality, though there are several exceptions to this generalization.

65. It is the belief of the writer that the possession of toughness is a very desirable quality in an airplane spar as it will be deformed to a noticeable extent before it fails, the incipient failure may be seen, and the member reinforced. With spars of the other type, failure is likely to be sudden and without warning. This difference in type of failure was quite noticeable in the tests, some spars failing almost without warning, the only indication of impending failure being that the deflection was increasing rapidly, and even this could not have been seen unless it had been for the constant scrutiny of the deflectometer by one of the test observers. In other cases, although the deflection readings and the slow increase of load indicated that the maximum load was being approached, it took some time before the axial load began to decrease or a local compression or other failure caused the spar to collapse.

66. Comparison of the relative toughnesses of the spars and their types of construction indicates that inherent strength against torsion is necessary for, though not an absolute guarantee of toughness.

UNIT STRESSES

67. One of the most important quantities to be determined in the study of tests like those made in this study, is the maximum unit stress developed in the tests by each spar. In practice it is impossible to determine the true maximum stresses developed as many of the spars were stressed beyond their elastic limits. For purposes of study, however, this fact is neglected and the stresses computed upon the assumption that the material retained perfect elasticity up to failure. This assumption makes it possible to obtain a unit stress value which, though not the true unit stress is comparable to the modulus of rupture, and can be used in the same manner.

68. There are two of these moduli that are of special interest. The first is that obtained from the ordinary formula for a beam subjected to combined axial and bending loads:

$$f = \frac{P}{A} + \frac{M_c}{I}$$

In applying this formula, it is necessary to take into account the secondary bending due to the axial load multiplied by the deflection, the latter quantity being that measured in the test. This value is referred to in this report as the maximum unit stress at failure. The second value of this moduli is obtained by considering the beam as a truss, at least in so far as its resistance to bending is concerned. To obtain this value, the axial load is divided between the chords and web in proportion to their sectional areas. The moment is then divided by the effective depth of the spar to determine the load in each chord due to bending. These chord loads due to bending are added algebraically to the

chord loads due to the axial load to determine the total chord load, and the sum divided by the area of the chord. This procedure gives the average stress in the chords. For the compression chord, this result can be obtained by the use of the formula,

$$f=\frac{P}{A}+\frac{M}{A_c h}$$

where A_c is the area of the compression chord and h the effective depth of the spar.

69. Table 8 gives the computations of these quantities for all of the metal spars tested. The deflections listed in column 6 of this table are the corrected deflections that are given in column 2 of Table 6; these values are subject to various errors as was explained in paragraph 61, but these errors are of little importance in the computation of unit stress at failure. The change in this unit stress due to as much as 0.1 inch error in deflection is not as great as the probable error due to other causes. So far as use in design is concerned, it is less than the uncertainties regarding form factor and variation in the physical properties of the materials used. In the intensive study of various types of construction, it may become more important to use accurate deflection data to compute unit stresses, but that stage has not yet been reached in this study.

70. For making the computations of bending moment, advantage was taken of the fact that as each of the side loads was equal to one-tenth of the axial load, and they were applied at a distance of 22.5 inches from the ends of the spar, the total moment at the center of the spar could be obtained by multiplying the axial load at failure by the observed deflection at failure plus 2.25 inches. In the cases of the five metal spars tested with eccentrically applied loads, the eccentricity of the application of load was allowed for by modifying the figure used for the deflection as shown in Table 8a.

71. Some engineers have tried to use the unit stress developed as a measure of merit for comparing spars, but the results in Table 8 show that this can not be done. So far as the spars of any one type are concerned, it is true that the strength-weight ratios and maximum unit stresses developed vary in about the same proportion. This, however, is not at all necessarily true as between spars of different types, as one type may have the material so effectively located that it will not be highly stressed until the beam is carrying a heavy load, while the other may be just as highly stressed under a small load. For example, spar 11B was subjected to a stress of 43,455 pounds per square inch when the axial load was only 16,250 pounds, while spar 20B was stressed to only 42,520 pounds per square inch when the external load was 23,510 pounds. The comparison of the unit stresses for these two types alone should demonstrate the futility of trying to rate designs by the unit stresses developed, though plenty of similar examples could be obtained from the data of Table 8.

72. The chief use of the unit stress values to the designer is to permit him to design a structure that will have the required strength without an excessive reserve. For this purpose, he is not so much interested in the unit stress computed after the test, using the observed deflection just before failure, as in the unit stress he must not exceed in his computations, the deflection used being also that which he computes from the properties of the spar section. In designing a spar for the loading used in these tests, or any similar loading, the designer would have to decide whether to use the beam formula $f+P/A+M_c/I$, or the truss formula $f=P/A+M/A_c h$. In either case, the designer will have to proceed by trial, selecting a trial section and then computing the unit stresses involved and comparing them with the allowable values. When he has selected the dimensions of the section of his trial design, he knows the values of P, A, A_c, I, and h, or can compute them from the dimensions chosen. The value of M is not known as it depends in part on the properties of the spar section selected.

TABLE 8.—*Unit stresses at failure*

Spar	Ult. load P	A	P/A	Measured δ	$\delta+2.25$	M	c_o/I	Mc_o/I	$f=\frac{P}{A}+\frac{Mc_o}{I}$	A_eh	$\frac{M}{A_eh}$	$f=\frac{P}{A}+\frac{M}{A_eh}$
1	2	3	4	5	6	7	8	9	10	11	12	13
10A	19,375	1.471	13,170	0.701	2.951	57,180	0.3404	19,440	32,610	1.925	29,700	42,870
10B	20,185	1.471	13,710	.950	3.200	64,592	.3403	21,960	35,670	1.925	33,540	47,250
10C	15,900	1.409	11,280	.700	2.950	46,905	.3307	15,500	26,780	2.200	21,340	32,620
10D	17,010	1.409	12,060	.661	2.911	49,516	.3307	16,350	28,410	2.200	22,510	34,570
10E	16,790	1.135	14,790	.760	3.010	50,538	.3371	17,020	31,810	2.200	22,970	37,760
11A	15,860	1.228	12,910	1.450	3.700	58,680	.4747	27,860	40,770	2.762	21,250	34,160
11B	16,250	1.228	13,230	1.670	3.920	63,700	.4747	30,225	43,455	2.762	23,060	36,290
12A	15,875	1.700	9,340	.630	2.880	45,720	.3323	15,200	24,540	--------	--------	--------
12B	19,240	1.700	11,320	.800	3.050	58,682	.3323	19,510	30,830	--------	--------	--------
13A	14,375	1.600	8,985	.611	2.861	41,127	.3789	15,585	24,570	--------	--------	--------
13B	16,425	1.600	10,265	.800	3.050	50,096	.3789	18,985	29,250	--------	--------	--------
14A	15,500	1.254	12,360	.510	2.760	42,780	.3480	14,890	27,250	2.965	14,440	26,800
14B	18,500	1.214	15,240	.719	2.969	54,927	.3571	19,600	34,840	2.894	18,980	34,220
14C	22,100	1.516	14,580	.800	3.050	67,405	.2877	19,390	33,970	3.683	18,295	32,875
14D	19,690	1.477	13,330	.650	2.900	57,101	.2925	16,695	30,025	3.624	15,770	29,100
15A	19,290	1.160	16,630	.985	3.235	62,403	.3600	22,470	39,100	3.220	19,370	36,000
16A	18,500	1.261	14,660	.960	3.210	59,385	.3147	18,690	33,350	3.505	16,950	31,610
16B	18,050	1.247	14,480	1.000	3.250	58,700	.3067	17,990	32,470	3.552	16,510	30,990
16C	18,750	1.350	13,890	1.085	3.335	62,630	.2892	18,080	31,970	3.800	16,450	30,340
16D	14,850	1.173	12,650	1.150	3.400	50,500	.2840	14.340	26,990	4.059	12,440	25,000
16E	14,960	1.166	12,830	1.280	3.530	52,850	.2783	14,720	27,550	4.326	12,210	25,040
16F	16,740	1.188	14,090	.665	2.915	48,800	.3075	15,000	29,090	3.473	14,040	28,130
16G	20,150	1.355	14,870	1.180	3.430	69,110	.2920	20,180	35,050	3.785	18,260	33,130
16H	19,900	1.386	14,350	1.073	3.323	66,130	.2850	18,850	33,200	3.878	17,050	31,400
17A	16,170	1.000	16,170	1.014	3.264	52,779	.4664	24,610	40,780	2.570	20,520	36,690
17B	15,300	1.000	15,300	1.000	3.250	49,725	.4950	24,610	39,910	2 498	19,900	35,200
18A	15,320	1.319	11,615	.950	3.200	49,024	.4380	21,470	33,085	1.653	29,665	41,280
18B	19,050	1.434	13,285	1.015	3.265	62,198	.3870	24,070	37,355	2.200	28,275	41,560
19A	14,580	.772	18,890	[1]–.891	1.359	19,810	.5130	10,160	29,050	2.520	7,860	26,750
19B	18,440	.782	23,580	[1]–.942	1.308	24,120	.5400	13,020	36,600	2,590	9,310	32,890
20A	19,740	1.420	13,900	1.172	3.422	67,550	.3025	20,425	34,325	3.875	17,430	31,330
20B	23,510	1.350	17,420	1.195	3.445	80,990	.3099	25,100	42,520	3.667	22,090	39,510
21A	20,180	1.589	12,700	.860	3.110	62,760	.2516	15,790	28,490	3.807	16,490	29,190
21B	20,980	1.496	14,020	1.290	3.540	74,270	.2657	19,750	33,770	3.660	20,290	34,310
21C	20,450	1.496	13,670	.845	3.095	63,290	.2657	16,820	30,490	3.660	17,290	30,960
21D	21,920	1.496	14,650	1.005	3.255	71,350	.2657	18,970	33,620	3.660	19,490	34,140
22A	18,450	1.127	16,370	.900	3.150	58,118	.2827	16,425	32,795	3.939	14,750	31,120
22B	19,800	1.219	16,240	1.750	4.000	79,200	.2851	22,580	38,820	3.817	20,750	37,000
23A	17,580	1.240	14,180	1.010	3.260	57,360	.5270	30,230	44,410	2.190	26,180	40,360
23B	15,760	1.061	14,850	.900	3.150	49,680	.5584	27,720	42,570	2.190	22,670	37,520
30A	20,800	.416	50,000	.936	3.186	66,269	.8840	58,650	108,650	1.208	54,800	104,800
30B	17,230	.416	41,500	.713	2.963	51,052	.8840	45,150	86,650	1.208	42,200	83,700
31A	14,125	.469	30,120	[1].209	2.459	34,720	.8445	29,320	59,440	1.464	23,710	53,830
31B	16,700	.469	35,610	[1].513	2.763	46,190	.8684	40,100	75,710	1.428	32,310	67,920
31C	22,120	.657	33,700	.880	3.130	69,240	.6447	44,640	78,340	1.988	34,830	68,530
32A	17,250	.459	37,580	1.070	3.320	57,270	.9178	52,600	90,180	1.338	42,810	80,390
32B	19,660	.459	42,830	1.110	3.360	66,058	.9366	61,870	104,700	1.315	50,240	93,070
40A	15,270	.835	18,300	[1].527	2.777	42,400	.4473	18,960	37,260	2.882	14,710	33,010
40B	18,300	1.497	12,220	1.813	4.063	74,400	.3355	24,950	37,170	4.300	17,300	29,520

[1] Corrected for eccentric application of load. See Table 8a.

TABLE 8a.—*Computation of net deflections—eccentrically loaded spars*

Spar	δ_o	e	Camber	δ
19A	0.750	0.891	0.750	−0.891
19B	.845	1.037	.750	−.942
31A	.608	.399	--------	.209
31B	.900	.387	--------	.513
40A	1.270	.743	--------	.527

δ_o = Observed deflection at failure.
e = Eccentricity of axial loading.
δ = Effective deflection = δ_o − e − camber.

73. The primary moment is known, but the designer must compute the secondary moment which is the product of the axial load and the deflection. As the axial load is known, the difficulty comes in computing the latter. No satisfactory method has yet been developed for computing the deflection of a truss under combined axial and bending loads, but such a method has been worked out for beams, and is discussed in Chapter II of "Airplane Design." In this study, it was decided to treat the trusses as though they were beams, using the beam formulas. This made it necessary to decide upon the values of EI to be used. In the discussion above on the stiffness of the spars, three values of EI were considered; E_sI_{xx} determined from the dimensions of the cross-section and the standard value of E for the material; EI_{xx} determined from a test in simple bending; and EI_{mt} determined from the test under combined bending and axial loads. In working out new designs, the designer would tend to use E_sI_{xx}, but the results of the tests made to date indicate that it would be better if he should modify that value in order to approach more closely the true effective EI for his spar. At present, it is not practicable to state how this should be done in all cases, but for the purpose of studying the test results, it was considered better to compute the deflections with one of the effective EI values determined by test. While the EI_{xx} values can be accurately determined as the lower portions of the load deflection curves under simple bending loads are straight lines, the EI_{mt} values are somewhat unreliable owing to the curvature of the lower part of the load deflection curve and the consequent difficulty in determining the zero error. Because of this, and the fact that the designer developing a new type of spar is more likely to be able to make tests in simple bending than under combined load, the values of EI_{xx} were chosen for use in the unit stress computations. A further reason for using these

values was that they had been computed for the spars with eccentric application of the axial load while the EI_{mt} values for these spars had not been computed.

74. Table 9 gives the computations of unit stress at maximum load which might be called the predictable unit stresses as they are based on the computed deflection corresponding to the EI value determined in a preliminary test. It might be said that unit stresses computed in this manner were of little value in interpreting the tests, as they are not based upon the actual deflections and are therefore obviously incorrect. That they are not the true average and maximum unit stresses at maximum load is admitted, but they are the kind of stresses which must be specified for use in new designs until some one develops a method of computing unit stresses and deflections beyond the proportional limit. Thus for the purpose of selecting a value for the allowable unit stress for use in design the unit stresses of Table 9 are of much more value than those of Table 8.

75. The results listed in Tables 8 and 9 show that the maximum unit stresses developed in the duralumin spars varied from about 24,000 to about 44,000 pounds per square inch and the variation in the case of the steel spars was even greater, due to the differences in heat-treatment and chemical characteristics of the steels used. It is evident from a study of these tables that it is not possible to derive general rules at the present time for the design of metal spars, but that special rules must be developed for each general type. In the following pages this will be done so far as possible for the duralumin box and duralumin channel truss types, as they are the only ones showing sufficiently high strength-weight ratios and of which enough spars have been tested to make the attempt worth while. No study of this type would be justified for the dumb-bell, hourglass, and I-beam types in duralumin or the steel channel truss type, until more spars of these types have been tested and better results have been obtained. These types show promise of successful development, but until they fulfill this promise, they need not be studied intensively. Certainly no intensive study of the types which showed up poorly with the object of developing rules for design would be warranted. In addition to this study of the two best types with the object of developing design rules, the group of spars as a whole will be studied with the object of generalizing of the effects of various points of detailed design.

TABLE 9.—*Unit stress at failure*

Spar	P	P/A	$\frac{EI}{1000}$	j	δ	M	c_c/I	Mc_c/I	$f=\frac{P}{A}+\frac{Mc_c}{I}$	A_ch	$\frac{M}{A_ch}$	$f=\frac{P}{A}+\frac{M}{A_ch}$
1	2	3	4	5	6	7	8	9	10	11	12	13
10A	19,375	13,170	75,200	62.3	0.816	59,400	0.3404	20,220	33,390	1.925	30,830	44,000
10B	20,185	13,710	91,200	67.2	.672	58,980	.3404	20,075	33,785	1.925	30,640	44,350
10C	15,900	11,280	86,600	73.8	.534	44,270	.3307	14,620	25,900	2.200	20,100	31,400
10D	17,010	12,060	88,200	72.0	.567	47,920	.3307	15,830	27,890	2,200	21,760	33,820
10E	16,790	14,790	90,600	73.5	.540	46,840	.3371	15,770	30,560	2.200	21,290	36,080
11A	[1] 15,860	12,910	53,400	58.0	.995	51,470	.4747	24,430	37,340	2.762	18,630	31,540
11B	16,250	13,230	54,600	58.0	.995	52,730	.4747	25,020	38,250	2.762	19,080	32,310
12A	15,875	9,340	89,000	74.9	.516	43,910	.3323	14,600	23,940	--------	--------	--------
12B	19,240	11,320	83,450	65.9	.707	56,890	.3323	18,910	30,230	--------	--------	--------
13A	14,375	8,985	72,800	71.2	.583	40,720	.3789	15,430	24,415	--------	--------	--------
13B	16,425	10,265	76,000	68.0	.653	47,680	.3789	18,065	28,330	--------	--------	--------
14A	15,500	12,360	78,300	71.1	.585	43,940	.3480	15,290	27,650	2.965	14,830	27,190
14B	18,500	15,240	96,000	72.0	.567	52,120	.3571	18,600	33,840	2.894	18,000	33,240
14C	22,100	14,580	106,700	69.5	.620	63,430	.2877	18,250	32,830	3.683	17,220	31,800
14D	19,690	13,330	133,500	82.4	.413	52,430	.2925	15,340	28,670	3.624	14,470	27,800
15A	19,290	16,630	81,100	64.8	.735	57,640	.3600	20,750	37,380	3.220	17,900	34,530
16A	18,500	14,660	77,500	64.7	.741	55,330	.3147	17,410	32,070	3.505	15,790	30,450
16B	18,050	14,480	83,700	68.1	.649	52,330	.3067	16,050	30,530	3.552	14,730	29,210
16C	18,750	13,890	87,100	68.2	.649	54,350	.2892	15,720	29,610	3.800	14,300	28,190
16D	14,850	12,650	79,900	73.4	.540	41,440	.2840	11,770	24,420	4.059	10,210	22,860
16E	14,960	12,830	72,300	69.5	.617	42,890	.2783	11,940	24,770	4.326	9,910	22,740
16F	16,740	14,090	78,000	68.2	.649	48,530	.3075	14,920	29,010	3.473	13,970	28,060
16G	20,150	14,870	84,600	64.8	.735	60,150	.2920	17,560	32,430	3.785	15,890	30,760
16H	19,900	14,350	87,400	66.2	.697	58,650	.2850	16,720	31,070	3.878	15,120	29,470
17A	16,170	16,170	60,650	61.2	.857	50,240	.4664	23,420	39,590	2.570	19,560	35,730
17B	15,300	15,300	61,300	63.3	.783	46,405	.4950	22,980	38,280	2.498	18,580	33,880
18A	15,320	11,615	69,800	67.5	.666	44,673	.4380	19,550	31,165	1.653	27,000	38,675
18B	19,050	13,285	80,100	64.8	.735	56,920	.3870	22,010	35,295	2.200	25,885	39,170
19A-2	14,580	18,890	33,360	47.8	([2])	23,770	.5130	12,190	31,080	2.520	9,433	28,323
19B	18,440	23,580	31,420	41.3	([2])	37,390	.5400	20,190	43,770	2.590	14,440	38,020
20A	19,740	13,900	93,900	69.0	.630	56,850	.3025	17,200	31,100	3.875	14,670	28,570
20B	23,510	17,420	91,400	62.4	.813	72,010	.3099	22,320	39,740	3.667	19,640	37,060
21A	20,180	12,700	93,400	68.0	.668	58,890	.2516	14,820	27,520	3.807	15,470	28,170
21B	20,980	14,025	87,400	64.5	.748	62,900	.2657	16,720	30,745	3.660	17,175	31,200
21C	20,450	13,670	100,100	69.9	.608	58,450	.2657	15,520	29,190	3.660	15,970	29,640
21D	21,920	14,650	97,500	66.7	.684	64,310	.2657	17,100	31,750	3.660	17,570	32,220
22A-2	18,450	16,370	74,350	63.5	.779	55,890	.2827	15,800	32,170	3.939	14,190	30,560
22B	19,800	16,240	82,900	64.7	.742	59,240	.2851	16,890	33,130	3.817	15,510	31,750
23A	17,580	14,180	68,100	62.2	.816	53,990	.5270	28,450	42,630	2.190	24,650	38,830
23B-2	15,760	14,850	62,700	63.1	.789	47,900	.5584	26,750	41,600	2.190	21,870	36,720
30A	20,800	50,000	89,000	65.4	.721	61,800	.8840	54,630	104,630	1.208	51,160	101,160
30B	17,230	41,500	93,100	73.5	.540	48,070	.8840	42,490	83,990	1.208	39,790	81,290
31A	14,125	30,120	72,400	71.6	([2])	32,690	.8445	27,610	57,730	1.464	22,330	52,450
31B	16,700	35,610	74,100	66.6	([2])	40,460	.8684	35,140	70,750	1.428	28,330	63,940
31C-2	22,120	33,700	97,100	66.3	.695	65,140	.6447	42,000	75,700	1.988	32,770	66,470
32A-2	17,250	37,580	89,000	71.8	.568	48,610	.9178	44,670	82,250	1.338	36,330	73,910
32B	19,660	42,830	87,000	66.5	.693	57,860	.9366	54,200	97,030	1.315	44,000	86,830
40A	15,270	18,300	45,500	54.6	([2])	34,510	.4473	15,440	33,740	2.882	11,980	30,280
40B	18,300	12,220	54,300	54.4	1.191	62,960	.3355	21,120	33,340	4.300	14,640	26,860

[1] EI for 11A by proportion from EI_{mt} of beam and EI value of 11B $54.6\times\frac{51.6}{52.8}=53.4$

[2] See Table 9a for computation of M taking eccentricity of each fitting into account.

TABLE 9a.—*Computation of bending moments eccentrically loaded beams*

Basic Formula $M_{L/2}=\dfrac{M_1}{\mathrm{Cos}\,\frac{L}{2j}}-\dfrac{W\sin\frac{a}{j}}{\mathrm{Cos}\,\frac{L}{2j}}$

Special Formula $M=P\left[\dfrac{0.1\,j\sin\frac{22.5}{j}-e}{\mathrm{Cos}\,\frac{48}{j}}\right]=P\,B$

this case where $B=\dfrac{A-e}{\mathrm{Cos}\,48/j}$

$A=0.1\,j\sin\dfrac{22.5}{j}$

and e = Eccentricity of application of axial load.

	Spar					Notes
	31A	31B	40A	19A-2	19B	
P	14,125	16,700	15,270	14,580	18,440	From Table 1.
j	71.6	66.6	54.6	47.8	41.3	From Table 9.
e	.399	.387	.743	.891	1.037	From Table 8A.
$\frac{22.5}{j}$	.314	.338	.412	.471	.545	
$\frac{48}{j}$	.670	.720	.878	1.004	1.162	
$\sin\frac{22.5}{j}$	.30906	.33160	.40045	.45378	.51842	
$\cos 48/j$	.78382	.75181	.63870	.53694	.39751	
$A=0.1\,j\sin\frac{22.5}{j}$	2.2129	2.2085	2.1865	2.1691	2.1411	
A−e	1.8139	1.8215	1.4435	1.2781	1.1041	
B	2.3141	2.4228	2.2600	2.3803	2.7776	
M=PB	32,69	40,460	34,510	34,705	51,220	

For the type 19 you assume true M = value above less 0.75 P.

M = 23,770 37,390.

TYPES OF FAILURE

76. Owing to the number of types of spars tested, there were many different types of failure, but most of the failures can be considered as belonging to one of four classes. Twenty-one of the 49 spars failed by lateral buckling of the compression chord. Eleven failed in joints between parts of the spars. Nine failed by crushing of a short length of the compression chord acting as a short column. Five failures were due to excessive bending of sections of the compression chord as long columns. The remaining three spars failed in other ways. The character of the failure of each spar is indicated in the second column of Table 10 by the following notations:

- L. B. Lateral buckling of the compression chord.
- S. Separation of the component parts of the spar due to joint failures.
- S. C. Crushing of the compression chord acting as a short column.
- L. C. Bending of a section of the compression chord as a long column.
- W. Failure of the web members.
- D. Excessive deflection accompanied by refusal to carry more load.
- F. Failure of end fitting.

77. The records regarding the types of failure are usually very brief, and in some cases not very clear now that the details of the individual tests have been largely forgotten, although at the time they were thought to be adequate. This shows the necessity of making specially careful and complete notes on the original data sheets regarding the character of the failure, a practice that will be followed in the future. In some cases, it is hard to determine which of the classes of failure that of a given spar should be listed under. This is particularly true in some of the cases where the failure was on the border line between two of the general types into which they are divided above.

LATERAL BUCKLING

78. The spars that failed by lateral buckling of the compression chord formed by far the most numerous class, including all the spars of types 11, 12, 13, 14, 17, and 23, and some of types 16, 18, 19, 20, 31, and 21. This kind of failure was typical of spars with narrow chord members, or with webs made up of a single sheet of material near the plane of symmetry. All of the I-beams, types 12 and 13, the plate girders, type 14, and the dumb-bells, types 17 and 23, failed in this manner. The boxes and channel trusses were relatively free from this kind of failure. One spar of the hour-glass type which has the web members close together at mid-height showed a failure of this kind.

79. In many cases in which the failure was in this manner, the spar remained practically straight until a load very close to the maximum carried was reached. The upper and lower portions of the compression chord then began to bow in opposite directions, or, if two points of lateral support were provided, the central third bowed out in one direction, and the end thirds in the opposite direction. Failure then came suddenly, the bowing out increasing rapidly, and

usually being accompanied by a complete compression failure of the chord near the center of the bow. When the failure occurred in this manner, there was usually little plastic deformation until the ultimate load was closely approached. This kind of failure was most common in spars that lacked inherent stiffness against torsion like the plate girder and I-beam types.

80. In the cases of the spars that failed by lateral buckling, although they were of types like the channel truss or the box which have inherent strength against torsion, the bowing usually became evident long before failure occurred and the final failure was not experienced until after much plastic deformation. It seemed that although the compression chord might have a tendency to buckle laterally, it was prevented from doing so by the stiffening effect of the tension chord aided by the web members, and the torsional stiffness of the spar. On the whole, those spars which failed by lateral buckling with little plastic deformation showed low strength-weight ratios, while those which failed by lateral buckling after considerable deformation, showed high ones. Four out of the five spars with the lowest strength-weight ratios belonged to the first class, and the spar with the highest strength-weight ratio fell in the second.

81. Spars 13A and 12B in Figure 12 show typical examples of the lateral buckling of I-beam spars, the former when tested with a single and the latter with two points of lateral support. After removing the spars from the testing jig, most of the deformation disappeared as is shown by the pictures of the types 12 and 13 spars in the same figure which were taken after the tests. In some cases, however, the load was applied until the buckling of the compression chord resulted in the complete collapse of a short length as shown by spar 14B in its picture appearing in Figure 12. Figure 13 shows another picture of spar 14B after it had failed but had not been removed from the test jig. The pictures of these spars in the test jig all show the lateral buckling very clearly, but there is not a sign of it in the pictures of the spars after the load had been removed.

82. The cause of lateral buckling failure is an interesting study, and as yet it has not been fully determined. The obvious way to attack it is to compute the strengths of the compression chords of the spars tested as columns, and determine the unsupported lengths that are the maxima at which they could be expected to carry the loads they actually were carrying at failure. Figures for such a study are given in Table 10.

83. In Table 10 the values of P/A are the average unit stresses in the compression chords at failure as obtained from column 13 of Table 8. Where, as in the cases of several of the box spars the computed maximum stress at failure in column 10 of Table 8 was less than the computed average stress from column 13, or no average stress was computed, as in the cases of the I beams, the value from column 10 was used instead of that from column 13. Such cases are indicated in Table 10 by a reference number after the value of P/A. These values were used as they were considered much more reliable than the computed average stresses. From the value of P/A used, the corresponding slenderness ratio was computed from Euler's formula. Multiplying the slenderness ratio so obtained by the radius of gyration of the compression chord about the major axis of the whole spar gives what might be called the effective length of the compression chord against lateral failure under the unit stress imposed by the load causing failure. These effective lengths are compared with the actual distances between points of lateral support in the test.

TABLE 10.—*Study of lateral buckling—Spars tested with single lateral support*

Spar 1	Failure 2	P/A 3	L/P 4	P_{ey} 5	L_e 6	L_a 7	L_e/L_a 8	Case 9
10A	S.	[1] 32,610	55.0	0.830	45.7	48.0	0.95	4
10B	S.	[1] 35,670	52.6	.830	43.7	48.0	.91	4
10C	S.	[1] 26,780	60.7	.830	50.4	48.0	1.05	2
10D	S.	[1] 28,410	58.9	.830	48.9	48.0	1.02	2
10E	S.	[1] 31,810	55.6	.830	46.1	48.0	.96	4
11A	L. B.	34,160	54.8	.602	33.0	48.0	.69	1
12A	L. B.	[1] 24,540	64.7	.626	40.5	48.0	.84	1
13A	L. B.	[1] 24,570	64.6	.625	40.4	48.0	.84	1
14A	L. B.	26,800	61.8	.631	39.0	48.0	.81	1
14D	L. B.	29,100	59.4	.676	40.1	48.0	.84	1
15A	L. C.	36,000	52.4	.858	45.0	48.0	.94	4
16A	S. C.	31,610	55.8	1.046	58.4	48.0	1.22	2
16B	S. C.	30,990	56.4	1.212	68.4	48.0	1.42	2
16C	S. C.	30,340	57.0	1.228	70.0	48.0	1.46	2
16D	L. C.	25,090	62.7	1.066	66.8	48.0	1.39	2
16E	L. B.	25,040	62.8	1.090	68.5	48.0	1.43	3
16F	L. C.	28,130	59.2	1.228	72.6	48.0	1.51	2
16G	S.	33,130	54.6	1.218	66.5	48.0	1.38	2
16H	S.	31,400	56.0	1.231	68.9	48.0	1.44	2
18A	L. B.	41,280	48.9	.636	31.1	48.0	.65	1
18B	S. C.	[1] 37,355	51.4	.828	42.6	48.0	.89	4
19A	L. B.	26,750	61.9	1.421	88.0	48.0	1.83	3
19B	W.	32,890	55.8	1.616	90.1	48.0	1.88	2
20A	F.	31,330	56.1	.781	43.9	48.0	.91	4
20B	L. B.	39,510	49.9	.792	39.6	48.0	.83	1
21A	S.	[1] 28,490	58.9	.864	50.9	48.0	1.06	2
21B	S.	[1] 33,770	54.1	.886	47.9	48.0	1.00	4
21C	L. B.	[1] 30,490	56.9	.886	50.4	48.0	1.05	3
21D	S. C.	[1] 33,620	54.2	.886	48.0	48.0	1.00	2
22A	S.	31,120	56.3	1.008	56.7	48.0	1.18	2
22B	S.	37,000	51.6	.997	51.5	48.0	1.07	2
23A	L. B.	40,360	49.5	.793	39.2	48.0	.82	1
31A	L. B.	53,830	72.9	.707	51.5	48.0	1.07	3
31C	L. C.	68,530	64.4	.806	51.9	48.0	1.08	2
40B	D.	29,520	59.0	1.219	71.9	48.0	1.50	2
11B	L. B.	36,290	53.1	.602	32.0	32.0	1.00	3
12B	L. B.	[1] 30,830	57.7	.626	36.2	32.0	1.13	3
13B	L. B.	[1] 29,250	59.2	.625	37.0	32.0	1.16	3
14B	L. B.	34,220	54.7	.616	33.7	32.0	1.05	3
14C	L. B.	32,875	55.8	.584	32.6	19.2	1.70	3
17A	L. B.	36,690	51.8	.736	38.2	32.0	1.19	3
17B	L. B.	35,200	52.9	.730	38.7	32.0	1.21	3
23B	L. B.	37,520	51.3	.793	40.7	32.0	1.27	3
30A	S. C.	104,800	52.2	.683	35.7	32.0	1.11	2
30B	S. C.	83,700	58.4	.683	39.9	32.0	1.25	2
31B	L. B.	67,920	64.9	.707	45.9	32.0	1.43	3
32A	S. C.	80,390	59.6	.710	42.4	32.0	1.32	2
32B	S. C.	93,070	55.4	.710	39.3	32.0	1.23	2
40A	L. C.	33,010	55.7	.768	42.8	32.0	1.34	2

[1] Computed maximum stress from formula P/A+Mc/I.

84. When a spar has failed by lateral buckling of the compression chord, it is reasonable to assume that the compression chord has acted like a pin ended column about the major axis of the spar, with an effective length not greater than the distance between lateral supports. In this case the ratio of the effective length of the compression chord, L_e, to the distance, L_a, between lateral support points will be less than 1. If the spar did not fail by lateral buckling it would be expected that the ratio would be greater than 1, which is to say that the length of the compression chord under the unit stress in question that would not need lateral support would be greater than the distance at which the supports were actually spaced.

85. The above reasoning assumes that the unit stress P/A is constant between the points of lateral support. If the unit stress should vary, some allow-

ance would have to be made for the variation, as it is obvious that a pin ended column with an imposed unit stress varying to a maximum of n pounds per square inch may be longer for the same cross section than such a column with a uniform unit stress of n pounds per square inch. In the test spars under consideration those tested with two or more lateral supports may be considered to have the compression chords subjected to a uniform unit stress, as the variation in the central third of their lengths was small. The majority of the spars, however, were tested with a single lateral support, and over about half of the distance between the end pin and the lateral support at the center of the span the unit stress in the compression chord was changing rapidly. As the effective lengths of Table 10 were computed from the maximum unit stresses, these spars would not be expected to fail by lateral buckling of the compression chord until the ratio of L_e/L_a was somewhat less than 1. Study of the test results indicates that the maximum ratio of L_e/L_a at which lateral buckling should be expected would be about 0.85.

86. In Table 10 the spars are divided into four classes, depending upon whether they failed by lateral buckling or in some other manner and whether the ratio of L_e/L_a is less than or greater than 1. Spars that failed by lateral buckling with a ratio of L_e/L_a less than 1 are put in class 1. Spars with the same type of failure but a ratio of L_e/L_a greater than 1 are in class 3. Spars failing in other ways are put in class 2 if L_e/L_a is greater than 1 and in class 4 if L_e/L_a is less than 1. Classes 1 and 2 thus include the spars which failed in the manner that was to be expected from the ratio L_e/L_a, while those of classes 3 and 4 failed in a manner inconsistent with the value of L_e/L_a.

87. It is noticeable that of the spars tested with a single lateral support, none of those in class 1 showed ratios of L_e/L_a greater than 0.84, while none of those in class 4 showed ratios less than 0.89. If it were assumed that the decrease in unit stress near the ends of the spar reduced the ratio of L_e/L_a below which lateral buckling failure might be expected to occur from 1 to between 0.85 and 0.88, all of the spars listed in class 4 would go in class 2, while none of those now in class 1 would have to be placed in class 3. This would mean that of the spars tested with a single lateral support, only four, 16E, 19A, 21C, and 31A, did not fail in the manner indicated by the ratio L_e/L_a. Of these four spars, 19A and 31A were tested with eccentric application of the axial load, and this may have something to do with the anomalous values of L_e/L_a but this can not be stated definitely. In the case of 19A there is the further factor that the two tubes forming the greater part of the compression chord were joined together by a light channel, and it is probable that it was insufficiently stiff to force the tubes to act as a unit. The result was that the effective radius of gyration of the compression chord was much less than was computed, being little more than the radius of gyration of one of those tubes. If this were the case, the failure by lateral buckling of those two tubes simultaneously would be largely explained. The only reasonable explanation that has been offered for the failure by lateral buckling of spars 16E and 21C is that due to unequal bearing on the end pins or some similar condition, the axial load was applied with an effective eccentricity perpendicular to the plane of the spar, causing a bending which eventually resulted in failure of the spar by lateral bending of the compression chord. One characteristic of the lateral buckling failures of spars 16E and 21C that should be noted is that their ratios of plastic to elastic deflection at maximum load were 158.7 and 40.8 per cent respectively, the value for 16E being the highest recorded. The ratios of plastic to elastic deflection at maximum load for the spars of class 1 varied from 5.1 to 40.5 with a mean of 25.4 per cent. This would indicate that the failures of these two spars, particularly 16E, were of somewhat different type than the typical lateral buckling failures due to the compression chords being loaded to a condition of elastic instability.

88. With the exception of spars 11A and 18A, all of those of class 1 showed ratios of L_e/L_a between 0.81 and 0.84. The two exceptions showed ratios of 0.69 and 0.65 respectively. The reason for this is not known with certainty, but is believed to be that the other spars of class 1 with the exception of 20B had single narrow webs that were unable to provide torsional stiffness and aid the compression chord when that member was highly stressed. The two spars with low ratios, however, had a good degree of torsional strength, 11A from the double web and stiffeners, and 18A by being made up as practically two tubes.

89. The tests with a single point of lateral support therefore indicate very strongly that, for this loading at least, if the unit stress per square inch in the compression chord is more than that corresponding to failure as an Euler strut when the distance between points of inflection is 0.85 the distance between lateral supports, and the torsional strength of the spar is small, the spar will fail by lateral buckling of the compression chord. If the spar has torsional strength, a higher unit stress is needed to cause lateral buckling. At a lower unit stress, failure, if it occurs, will be in some other manner.

90. It has been shown (see pages 221–22 of Pippard and Pritchard's "Aeroplane Structures") that when the axial load in a long strut varies, the strength of the strut can be estimated very closely by using the average load. In the spars tested the average value of P/A varied from about 84 to 89 per cent of the maximum, these figures being obtained by assuming that the unit stress in the compression chord due to bending varied from zero at the ends to a maximum at the side load points, between which points it remained constant, and that the ratio of unit stress due to the axial load on the spar to the total unit stress varied from 0.34 to 0.54 which were the extreme values for this quantity as determined from a study of Table 8. If, instead of computing the effective length L_e from the maximum unit stresses in the compression chords, average unit stresses varying from 84 to 89 per cent of the maxima had been used, the ratio of effective to actual lengths would have been from 0.92 to 0.96 instead of from 0.81 to 0.84. This would indicate that the dividing line between classes 1 and 2 of the spars in Table 10 should have been put at a value of L_e/L_a of about 0.94 instead of 0.85. However, all of the compression chords receive support from the webs and so can not act entirely

as independent columns, and it may well be that the small amount of torsional strength provided by the weakest of the webs of the spars tested was sufficient to reduce the critical ratio of L_e/L_s to about 0.85.

91. Though nearly all of the spars tested with a single lateral support failed in a manner consistent with the values of L_e/L_s, the same was not true of the spars tested with more than one lateral support. Most of these spars failed by lateral buckling, but with ratios greater than 1. Two explanations can be given for this phenomenon. One is that as the distance between points of lateral support is so much less than when only one lateral support was used, the effective length L_e should have been figured by a formula like the straight line or parabolic formulas giving lower values than the Euler formula. The other is that on account of the clearance between the spars and the lateral supports, the spar deflected with a distance between true points of inflection greater than the distance between those supports. These possibilities are not inconsistent with each other, and the results may be due to a combination of them. An attempt was made to find a value of L_e/L_s for these spars which could be used as a corrected division point between the spars that failed in a manner consistent with the value of L_e/L_s and those which failed in a manner inconsistent with that value, like the ratio 0.85 to 0.88 for the spars with a single support, but none could be found.

92. The results of the tests with two or more lateral supports are not inconsistent with the theory expressed in paragraph 89, but they show that the character of the loading and also the slenderness ratio of the compression chord has a great influence on the likelihood of lateral buckling failure so that much more work will have to be done on the problem before it can be considered as completely solved.

SHORT COLUMN FAILURES

93. Nine of the spars failed by crushing of short lengths of the compression chord, namely: 16A, 16B, 16C, 18B, 21D, 30A, 30B, 32A, and 32B. This type of failure is one that reflects credit upon the efficiency of the design, though the properties of the material used may be such that the strength-weight ratio may not be so large. This is true because it indicates that all connections were adequate and that the unit stress was increased to a point where crushing occurred instead of buckling in the manner of a long column. In all of these spars, with the exception of 32A in which the strength-weight ratio fell to 1,207, the strength-weight ratios were good, being above 1,360, and in five cases above 1,400.

94. The spars of type 16 are dural channel trusses, the channels being of the type studied by Roy A. Miller in Air Corps Information Circular, Vol. VI, No. 598, "Compressive Strength of Duralumin Channels." In this report Mr. Miller computed from the results of his other tests the unit stresses that should cause the chord members of these spars to fail if considered as pin-ended columns, with their unsupported lengths equal to the distance between the joints connecting the webs to the chords. These unit stress figures are given in Table 11 along with the computed unit stresses in these channels from Table 8 and the ratios between the theoretical stresses of Mr. Miller and the computed stresses from Table 8.

TABLE 11.—*Unit stresses in channel chords, spars 16A, B, and C*

1	2	3	4	5	6
	Unit stresses			Ratios	
Spar	Theoretical	Average	Maximum	Average/Theoretical	Maximum/Theoretical
16A	29,820	31,610	33,350	1.060	1.118
16B	28,410	30,990	32,470	1.091	1.142
16C	29,250	30,340	31,970	1.037	1.093
Average				1.063	1.118

95. It can be seen from this table that the average unit stresses obtained are a little more than 6 per cent greater than Mr. Miller's theoretical value, which is a much better check than most of those obtained in this spar study. It is also to be noted that from character of the failure as a short column by crushing, one would expect the observed stress to be somewhat larger than the theoretical value, the latter being based upon the assumption that the failure would be in part at least by buckling.

96. Though the failure of spar 18B is recorded as being one of short column crushing, its action in test was such as to arouse the suspicion that the crushing was secondary and that the primary failure was due to lateral buckling. The chief indication of this was its ratio of L_e/L_s of only 0.89. The same may also be true of 21D, which showed a ratio of only 1. The other spars of this group all show fairly high ratios of L_e/L_s, the average for the eight, excluding 18B, being 1.24.

97. Both spars of type 30 showed high strength-weight ratios, this as well as the type of failure indicating that the material was efficiently used. The spars were channel trusses of heat-treated steel. The further development of this type has not yet been pushed on account of the fact that their narrowness made it necessary to test them with two lateral supports to prevent them from buckling laterally. That this was a wise precaution is shown by the L_e values for them, both of which are less than 48 inches. As it was, these spars were able to carry load until the crushing strength of the material was reached, spar 30B failing under a lower stress than 30A because the joints of the latter were riveted and those of the former welded, and the resulting deterioration of the material caused the compression chord to fail near a welded joint at a relatively low unit stress.

98. The spars of type 32 were welded steel trusses of tubing. Though the EI_{yy} values for these spars were rather low, it was first attempted to test them with a single lateral support, but on their beginning to show signs of lateral buckling of the compression chord, the test was stopped and both spars tested with two lateral supports. In this test they failed by crushing of the compression chords. In the case of 32B, this did not

occur near the center of the spar but near the ends, due probably to poor alignment of the parts of the spar in construction, or warping after welding.

99. Although the type of failure being considered is considered one of the best that can be obtained in a spar, the data on hand gives little hint as to how it can be secured with certainty. About the best the designer can do is to try to proportion his spars efficiently and hope for the best.

LONG COLUMN FAILURES

100. Four of the spars, 16D, 16F, 31C, and 40A failed due to buckling of the compression chord in the plane of the truss, the failure taking place over a single panel of the truss. One spar, 15A, had a similar failure except that the buckling was at right angles to the plane of the truss. It should be noted that all of these spars were trusses. In this type of failure it seemed that the compression chord acted as a long column between the truss joints. This would happen if at all only under relatively low unit stresses and one would expect spars failing in this manner to show rather poor strength-weight ratios. On the whole this expectation is realized, the strength-weight ratios obtained being 1,419 for 15A, which was deeper than allowed by the specification; 1,376 for 40A, but this spar was helped by eccentric loading; and 1,331 for spar 16F, which incidentally was designed with a specially shallow chord so it would fail in this manner. For the other two spars the strength-weight ratios were 1,230 for 16D and 1,194 for 31C. In all of these cases the maximum load could have been increased by either designing the compression chords with larger minimum radii of gyration, or by decreasing the length of the truss panels. The first method has the disadvantage, rather clearly shown by spar 11B, that the designer is in danger of having too much material near the neutral axis of the spar to permit an efficient design, the possible increase in unit stress being more than offset by the decrease in section modulus. The second method has the disadvantage that the extra weight of web members may offset the extra efficiency of the chords.

101. The plastic deflection (Table 8) of these spars was large owing to the considerable shortening of the length of chord member that buckled. At failure the average plastic deflection of the first four mentioned spars was 57 per cent of the elastic deflection. 40A is omitted from the average as it was listed with eccentrically applied load and EI_{mt} was not computed for it.

JOINT FAILURES

102. The failures of 11 spars were due to the failures of connections between the parts forming the spars. These were the five spars of type 10, and 16G, 16H, 21A, 21B, 22A, and 22B. Seven of these spars were therefore boxes, and four were duralumin channel trusses. The spars of type 10 all failed in the same manner, some of the rivets joining the webs to the cover plates first failing in tension. This suddenly doubled the unsupported length of the cover plates which immediately became long slender columns subjected to a higher unit stress than they could carry with their new unsupported lengths and they immediately buckled. This kind of failure is shown clearly in Figure 18, a close-up of the failure of spar 10A. In type 21 the edges of the cover plates were turned

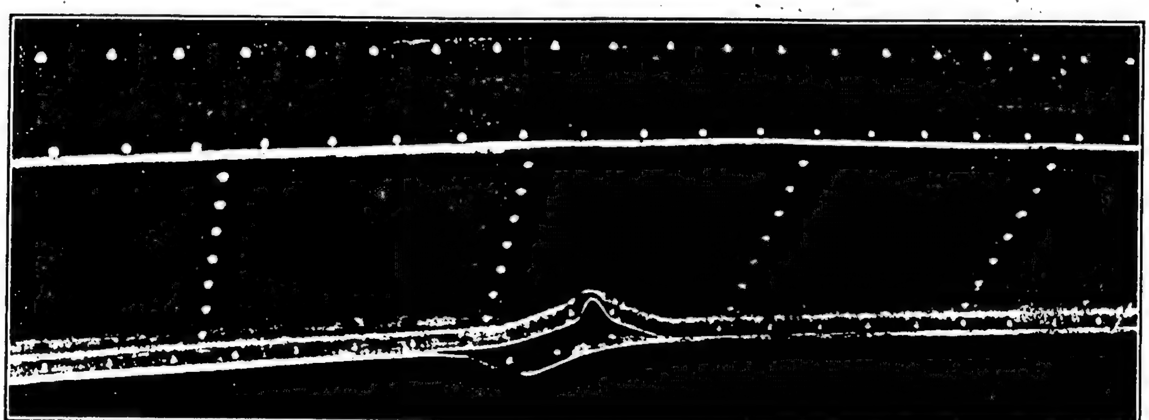

FIG. 18

down so the loss of support from the webs by failure of these rivets would not be so serious as the minimum radius of gyration of the cover plate alone was greatly increased. It is noteworthy that while the first two spars of type 21 failed in this manner the failure was not so sudden and 21B had a higher strength-weight ratio than any of the type 10 spars. Though the classification is such that the rivets are apparently to blame for the failure of the type 10 spars, this is not wholly the case, as the failure of these rivets was undoubtedly due to the desire of the thin cover plates to buckle. When the cover plates were made as shallow channels, they were so much stiffer that they did not have the same buckling tendency, and did not pull off the rivet heads as quickly as they did in type 10.

103. Spar 22B was a channel truss with the back of the compression chord reinforced by a flat strip riveted to it. This flat strip acted just like the cover plates of the type 10 spars and buckled away from the channel, pulling off the head of at least one rivet in doing so. This did not cause immediate failure, but at a little higher load, the two parts of the compression chord no longer reinforcing each other, both failed simultane-

ously along a short length. While the two parts seemed to fail simultaneously, it is probable that the flat strip failed first throwing extra load on the channel which was already too heavily loaded to be able to carry the increase.

104. The other three channel trusses failed by tearing away of web members at the joints connecting them to the chords. In all probability none of these web members would have failed if the load in them had been applied at their centroids, but as it was applied eccentrically, the bending caused extra tension at the free edges of the channels and failure resulted. In the case of spar 22A the failure was in the rivets, and was probably due to too close design of this member, the secondary shear due to the slope of the elastic curve of the spar and the axial load being neglected.

105. On the whole, a failure of this kind is an indication that the type of spar being used has not been utilized to its possible efficiency, and this is also indicated by the better strength-weight ratios obtained from some of the boxes of types 20 and 21 as compared with those of type 10. In the case of the channel trusses, this does not seem to be the case as 16G has the best strength-weight ratio of all of the spars of types 16 and 22, but if the other features of this spar were used with a stronger web, it is probable that a better strength-weight ratio could be obtained.

106. All of the spars failing in this manner showed fairly high ratios of plastic to elastic deformation at maximum load, the values ranging between 4.8 and 110 per cent of the elastic deflection and the arithmetric mean being 50 per cent.

OTHER FAILURES

107. Three of the spars failed in ways not covered by the four types of failure mentioned above. The end fittings of spar 20A were weak and poorly connected to the spar proper. The result was that the fitting failed and the end of the spar crushed on the uneven bearing provided it by the fitting. This really could not be considered a spar failure, and the results with 20A should be given little if any weight in comparing the types.

108. Spar 40B was subjected to load until it refused to carry more. The test could not be carried on to complete failure, however, as the spar deflected until it pressed against the jig, and the additional support allowed it to begin to increase the load it carried. The failure is therefore classed as excessive deflection in the plane of the truss. Its strength-weight ratio was so low that it has not been considered worth-while to study this type more thoroughly, even to determine with certainty the character of its failure.

109. Spar 19B failed by buckling of the compression web members between the end pins and the points of application of the bending load. This failure was thus very similar to that of spar 22A, the fundamental cause of the failure being lack of strength in the web members. In this case also the weakness of the web was probably due to neglect or underestimation of the increase in the shear due to the slope of the elastic curve of the spar near the ends, or possibly to overestimation of the coefficient of restraint of the web members and their ability to carry loads of compression.

DESIGN OF SPARS

110. The wide variation in unit stress values shown in Tables 8 and 9 and the lack of any correlation between unit stress and strength-weight ratio shows that special rules will be needed for the design of each type of metal spar. The determination of the proper rules for a type of construction requires many tests to determine the effects of each of the many variables affecting the result, and at the present time sufficient tests have not been made on any type of spars to permit formulation of complete design rules for that type. In the case of two types, however, several spars have been tested, and the data obtained from these tests is considered sufficient to permit the formulation of rules which, though not complete, will be of much assistance in design, and can be used in practice until new data shows what changes should be made in them. This section of this report is devoted to the derivation of tentative design rules for the two most promising types, the duralumin box and the duralumin channel truss.

DURALUMIN BOXES

111. The problems confronting the designer laying out a new design are chiefly the following, each of which will be discussed in turn with reference to the dural box in the following paragraphs:

(*a*) The allowable unit stress at design load.

(*b*) The stiffness factor or probable ratio of effective EI to E_sI_{xx}.

(*c*) The features required to prevent lateral buckling of the compression chord.

(*d*) The proper proportions for the cover plates or chord members.

(*e*) The proper proportions for the web.

(*f*) The determination of rivet spacing to prevent failure by separation of the parts of the spar.

(*g*) The proper design of fittings at points of application of concentrated loads.

112. In the series of tests made to date 11 duralumin box spars have been tested, 5 of type 10, 4 of type 21, and 2 of type 20. Four computations of unit stress at failure have been made for these spars, and tabulated in Tables 8 and 9. Not all of these four figures, however, are of use to the designer. Those of Table 8 are based upon the observed deflections at maximum load, and those of Table 9 on deflections at maximum load computed on the basis of the values of EI_{xx} obtained from cross-bending tests. For purposes of the formulation of design rules, it is considered that the figures shown in Table 8 can be neglected in favor of those of Table 9.

113. Two unit stress values are shown in Table 9 for each spar, one computed from the formula $f=P/A+M_y/I$, and the other from $f=P/A+M/A_ch$. The former is supposed to be the maximum stress in the compression chord, while the latter is supposed to be the average stress in that chord. It may be noticed, however, that for all of the spars of types 10 and 21, the "average" stress is larger than the maximum. This is an absurd result and a clear indication that at least one of the values is in error. The probability is that the average stress is the one in error, as

the entire cross section was taken account of in computing the maximum stress while the effect of the web was neglected in computing the average value. This being the case, it is considered that for duralumin box beams, the design should be based upon the computed maximum unit stress, and only those values, given in column 10 of Table 9 will be considered in determining on a unit stress value for use in design.

114. The unit stress values shown in column 10 of Table 9 for the 11 box spars tested are as follows: 10A, 33,390; 10B, 33,785; 10C, 25,900; 10D, 27,890; 10E, 30,560; 20A, 31,100; 20B, 39,740; 21A, 27,520; 21B, 30,745; 21C, 29,190; 21D, 31,750. The lowest value is 25,900 and the highest 39,740. The median value is 30,745. The lowest value is therefore 84.2 per cent of the median.

115. If it were desired to be ultraconservative, the allowable unit stress would have to be set near the lowest recorded value, at, say, 26,000 pounds per square inch. The writer considers, however, that spars 10C and 10D do not represent good design, as the cover plates did not have the edges turned down, and what is more important, the webs were very light and unable to give the cover plates adequate support. If the cover plates are adequately stiffened either by turning down their edges to form shallow channels, dishing them as done in type 20, or by connecting them to a rigid web, the spar should not fail at a load less than that which would give a computed maximum unit stress of 30,000 pounds per square inch. As in the case of wood boxes, the allowable unit stress undoubtedly varies with the proportions of the section, and the problem can not be considered solved until a method of computing the form factor has been evolved. Sufficient data for this purpose is lacking at present so it will be necessary to use a single figure for boxes of all proportions, though as soon as spars having some particular design feature in common show a well-defined tendency to fail under loads corresponding to some other unit stress, the value should be accordingly raised or lowered for this type.

116. In order to compute the maximum unit stress in a new spar, it is necessary to compute its deflection either directly or else indirectly by using the Newell formulas for combined bending and compression. In either case, a value of EI is needed. As was shown above, the effective EI as obtained from either cross-bending tests or combined load tests will be appreciably less than the value called E_sI_{gx} in this report. The designer must therefore have a correction factor by which to modify his value of E_sI_{gx} to obtain the EI value for him to use in his unit stress computations. For the 11 box spars under consideration, the values of EI_{xx}/E_sI_{gx} range from 0.820 to 0.994, the median being 0.916 and the mean 0.9155. The ratio EI_{mt}/E_sI_{gx} varies from 0.810 to 1.056, the median being 0.959 and the mean 0.935. Though the loading from which the EI_{mt} values were obtained is more like that to which the spars are subjected in service than the loading used to obtain EI_{xx}, the latter are considered more reliable for the reasons given in paragraph 73. It is believed wise therefore to give more weight to the EI_{xx} values. When the variations in the designs are considered, it is recommended that for design purposes a stiffness factor of 0.9 be used for duralumin box spars, this factor being applied as a multiplier to the value of E_sI_{gx} to obtain the effective EI value to be used in further computations.

117. Having selected values for the allowable unit stress and for the stiffness factor, it is of interest to determine the allowable loads based on these criteria and to compare them with the observed maximum loads. The computations for this comparison are given in Table 12. On account of the secondary bending it was not possible to compute the strength of these spars directly, but a load had to be selected, the unit stress resulting from that load computed, and the value compared with the specified allowable stress of 30,000 pounds per square inch.

TABLE 12.—*Computed strength of box spars*

	Spar										
	10A	10B	10C	10D	10E	20A	20B	21A	21B	21C	21D
E_sI_{gx}	91.8	91.8	94.5	94.5	92.7	103.3	100.8	113.7	102.1	102.1	102.1
$EI=0.9\ E_sI_{gx}$	82.6	82.6	85.0	85.0	83.4	93.0	90.7	102.3	91.9	91.9	91.9
FIRST TRIAL											
P	20,000	20,000	16,000	16,000	16,000	19,000	23,000	20,000	20,000	20,000	20,000
$j=\sqrt{\frac{EI}{P}}$	64.2	64.2	72.8	72.8	72.2	69.9	62.7	71.6	67.8	67.8	67.8
δ	0.750	0.750	0.550	0.550	0.565	0.612	0.795	0.573	0.660	0.660	0.660
M	60,000	60,000	44,800	44,800	45,050	54,400	70,000	56,460	58,200	58,200	58,200
A	1.471	1.471	1.409	1.409	1.135	1.420	1.350	1.589	1.496	1.496	1.496
c_c/I	0.3404	0.3404	0.3307	0.3307	0.3371	0.3025	0.3099	0.2516	0.2657	0.2657	0.2657
P/A	13,600	13,600	11,360	11,360	14,100	13,390	17,040	12,600	13,360	13,360	13,360
M_c/I	20,420	20,420	14,820	14,820	15,190	16,460	21,690	14,210	15,460	15,460	15,460
f	34,020	34,020	26,180	26,180	29,290	29,850	38,730	26,810	28,820	28,820	28,820
SECOND TRIAL											
P	17,650	17,650	18,350	18,350	16,400	19,100	17,800	22,400	20,800	20,800	20,800
j	68.4	68.4	68.0	68.0	71.2	69.8	71.4	67.6	66.5	66.5	66.5
δ	0.644	0.644	0.653	0.653	0.583	0.617	0.579	0.665	0.690	0.690	0.690
M	51,100	51,100	53,300	53,300	46,500	54,800	50,400	65,300	61,200	61,200	61,200
P/A	12,000	12,000	13,030	13,030	14,450	13,450	13,190	14,100	13,900	13,900	13,900
M_c/I	17,390	17,390	17,630	17,630	15,370	16,580	15,620	16,430	16,260	16,260	16,260
f	29,390	29,390	30,660	30,660	29,820	30,030	28,810	30,530	30,160	30,160	30,160
Computed P	18,030	18,030	17,950	17,950	16,500	19,070	18,530	22,000	20,680	20,680	20,680
Observed P	19,375	20,185	15,900	17,010	16,790	19,740	23,510	20,180	20,980	20,450	21,920
Margin of safety (per cent)	7.5	12.0	−11.4	−5.2	1.8	3.5	26.9	−8.3	1.5	−1.1	5.6

Average margin of safety (neglecting 20B) = 0.57 per cent. Including 20B = 2.96 per cent.
Average deviation (neglecting 20B) = 5.68 per cent. Including 20B = 7.38 per cent.

118. For the first trial, the axial load was assumed to be an integral number of thousands of pounds and a little less than the observed maximum load, the same figure being assumed for all spars having the same geometrical properties. The effective EI values were obtained by multiplying the values of E_sI_{ex} taken from Table 9 by the recommended stiffness factor 0.9. The deflection was obtained from the value of *j* computed from the effective EI and assumed P values and the curve of Figure 15. The remainder of the computations down to and including the value of *f*, the maximum computed unit stress, are conventional and need not be discussed further.

119. The axial load P used in the second trial was obtained by assuming that the unit stress varied in direct proportion to the axial load, and using this assumption to determine the axial load which would produce a maximum unit stress of 30,000 pounds per square inch. The results of the second trial were to give maximum unit stresses so close to 30,000 that the third trial load, computed from the second in the same manner as the second was computed from the first, was assumed to be the computed strength of the spar.

120. Comparison of the strength of the spar as computed on the recommended basis with the observed maximum loads shows that the computed strengths are, on the average, on the safe side by about one half of 1 per cent if spar 20B is neglected, but by about 3 per cent if that spar is included. The average deviation of the observed strengths from those computed is about 6 per cent if 20B is neglected and about 6 per cent if it is included. As this agreement is so good, and the spar which failed to carry the computed load by the largest margin failed by only 11.4 per cent, the results in Table 12 are considered sufficient justification of the recommended values of maximum unit stress and stiffness factor.

121. The fairly high margins of safety shown in Table 12 by spars 10A and 10B and particularly 20B indicate that when more is known about these types of spars, it may be possible to increase the unit stresses allowed in their design. All of the spars showing serious negative margins of safety incorporate what are now known to be defects in design, but until these defects and the methods of avoiding them have been studied more thoroughly it is considered better to be conservative.

122. Comparison of spar 20B with 11A and 11B would lead one to suspect that it would develop a higher unit stress than those of types 10 and 21, on account of the greater amount of material near the neutral axis of the spar which can give aid to the more highly stressed portions. In fact, the justice of classifying these spars with those of types 10 and 21 is open to serious question. The writer believes that further study of type 20 will show that a higher unit stress is allowable, and he would use a higher stress, probably 36,000 to 38,000 pounds per square inch, if designing spars for test, but would use 30,000 for spars to be used in actual airplanes until more study has been devoted to this type. The fact that spar 20A did not show a unit stress much higher than 30,000 would not influence him on this matter as that spar failed in test by crushing of the end fitting, and thus showed only that the spar could carry at least the unit stress value shown, but gave no indication as to how much higher stresses it would carry.

123. In promulgating the unit stress value of 30,000 and the stiffness factor of 0.9, it is implied that the spars should not only be of the box type, but should also be designed in such a manner as to take advantage of the lessons learned in the tests as to proportioning of members, etc.

124. One of the most important matters to be investigated in the design of such spars is the likelihood of lateral buckling of the compression chord. After the trial design has been completed, the compression chord should be investigated as an Euler column between points of lateral support, and if the ratio of L_e/L_a computed in the manner used in compiling Table 10 is less than 0.9 the design should be changed to obtain more lateral stiffness. In making the computation for L_e for a box spar, it will be advisable to use the maximum unit stress instead of the average stress, unless the latter proves to be the smaller. The discussion of Table 10 indicates that the maximum stress will give a more reasonable value, and its use will also simplify the computations, as it must be computed in any event in order to check the strength of the spar in the plane of its major axis. In practice, as the maximum allowable unit stress, 30,000, is known, and so is the distance between lateral supports of the compression chord, the minimum allowable value of the radius of gyration of the chord about the major axis of the spar can be computed directly and the design made in accordance.

125. In the 11 box spars tested, 3 types of chord member or cover plate were used. The type first used was the flat plate. In the first 2 spars tested this showed up quite well, but at maximum load it tore away from the web and failed. In the other 3 spars of type 10 the cover plates were made heavier and the webs lighter. The result was that while the minimum radius of gyration of the cover plate was increased, the stiffening from this cause did not offset the loss of stiffness due to the lighter web. This accounts for the failure of the spars at low unit stresses. The spar with flanged lightening holes gave the best results of this group of 3 spars, as the result of the lightening hole flanges was to practically double the number of points where the cover plate was stiffened by the web. It was the belief of those who observed the test that if the cover plates had had sufficient inherent stiffness the webs would have not been called on to stiffen the cover plates and the design with lightening holes would not have showed up as well. As a result of the tests on the spars of type 10 it was decided that in any further work with duralumin box spars the flat cover plate would not be used, and it is recommended that this policy be adhered to in all design work. Even though it is possible to design a spar with adequate strength with flat cover plates, it is believed that such a spar when it fails will do so by tearing away suddenly from the web, and that this is an indication of less than possible efficiency in the use of the material.

126. In the spars of type 21 the chords were made up of one or two pieces of sheet duralumin, the edges

of the outer sheet being turned down at right angles to form a shallow channel. The object of this was to increase the minimum radius of gyration of the chord and prevent failure by pulling away from the webs. On the whole, this device proved successful, and it is recommended that either this or some equivalent device be employed by all designers of box spars. There is insufficient data on hand at present to determine how much of the free edges of the cover plates of spars of type 21 should be turned down to provide stiffness, and this is one of the moot questions regarding design that should receive attention in the intensive study of this type of spar.

127. The possible methods of stiffening the compression chord of a dural box spar are numerous. Not only can the free edges be turned down to form a shallow channel, but one or more corrugations might be placed longitudinally in the sheet, or it might be dished in some manner. Type 20 shows a type of dishing in which the resulting chord member is a closed hexagonal section. The results of the tests on the two type 20 spars made to date are most favorable, and it may be that it represents the type of box that should be generally used. Only one of the tests, however, gave much light on the allowable unit stresses for this type, and there is as yet no data available as to the proper proportions for the chord members. One of the most urgent phases of spar research at present is to obtain more data on this subject.

128. There are two important phases to the design of the webs of duralumin box spars: The determination of the thickness of sheet to be used, and the decision as to the number and character of web stiffeners to be used. On neither of these points, however, do the tests give as clear information to the designer as is given regarding the allowable unit stresses in bending and compression.

129. Table 13 shows computations of the average unit shear stresses in the webs of the 11 box spars tested, no allowance being made for the shear resulting from the axial load and the slope of the elastic curve. As can be seen from the table, though the unit shear values are quite constant for each type, they vary greatly between types. The spars of type 21 showed a distinct tendency to fail through weakness of the webs, and in comparison with type 10 the shear values would lead one to expect this. On the other hand, type 20 showed no tendency to web failure, though the webs of these spars are subjected to twice as great a unit shear. The difference may be due, however, to the corrugations of the type 20 webs. All of the shear values are low, but the secondary shear, and also the part of the axial load carried by the web, is neglected.

130. The allowable stress in the webs undoubtedly depends upon the distance between the chord members and also that between stiffeners, but there is too little data available to make very specific recommendations for design. It is considered conservative design to have the distance between stiffeners somewhat less than the distance between the chords, and for the 6-inch depth of spar the unit shear stress should probably be kept below 4,000 pounds per square inch. The more complete determination of these points should be one of the objects of an intensive study of this type of spar.

131. All of the types of web stiffener used, channels joining both webs, alternate channels joining both webs and angles on the outer surfaces of the webs, continuous vertical corrugations, and large vertical corrugations spaced at 4 inches, and vertical channels joining both webs alternating with round flanged lightening holes seemed to be adequate, and no real advantage seemed to be obtainable from any of them, as compared to the others. It is believed, however, that the stiffeners joining both webs, though more expensive, are better than the other types, as they give the spar more strength against torsion and thus make failure by lateral buckling of the compression chord less likely. Channels joining both webs were used both at right angles to the axis of the spar and at 45° in the pattern of the web of a Warren truss, but as the latter showed no definite advantage in strength over the former and is a more expensive type of construction it is not recommended for use in practical design.

TABLE 13.—*Unit shear stresses in webs of duralumin box spaces*

	Spar										
	10A	10B	10C	10D	10E	20A	20B	21A	21B	21C	21D
h	5.912	5.912	5.90	5.90	5.90	4.944	4.944	5.8815	5.8815	5.8815	5.8815
t	0.06	0.06	0.050	0.050	0.050	0.022	0.022	0.042	0.042	0.042	0.042
s	6″	6″	5.250″	45°	5.25	.750″	(1)	4″	4″	5.25	5.25
A_w	0.71	0.71	.059	0.59	0.59	0.218	0.218	0.494	0.494	0.494	0.494
S	1,937	2,019	1,590	1,701	1,679	1,974	2,351	2,018	2,098	2,045	2,192
$f=S/A_w$	2,730	2,840	2,700	2,880	2,850	9,050	10,780	4,090	4,250	4,150	4,440

(1) Corrugations.

A_w = Area of web.
S = Primary shear.
s = Distance between stiffeners.
t = Thickness of web.
h = Height of web sheet (clear distance).

132. There were not enough tests run to determine any of the fine points regarding rivet spacing. It seems that a good rule to follow for the rivets connecting the web to the chord members is to so space them so that the maximum slenderness ratio of the chord on the length between adjacent rivets will be less than that of the spar as a whole about its minor axis. If a flat cover plate is used this criterion will put the rivets very close together, but the tests show that many rivets are needed with this type of cover plate. With a dished or shallow channel cover plate, the required rivet spacing will be much larger, and probably one that the designer will consider very reasonable.

133. The nature of the tests made in this study was such that they throw little light on the problem of the design of fittings for the application of load to the spar. The end fittings through which the axial load was applied were cut off before the spar was weighed, and were therefore made so heavy that there was little chance of their failure during test. This was intentional as a previous series of tests had shown that if the weight of such fittings was included in the spar weight used in making comparisons, these fittings would be made so light that they would often fail long before the main portion of the spar was heavily loaded, making the test of little or no value in determining the merits of the type of spar. The results of the tests on spar 20A are a good example of this. The side pull fittings did have to be carefully designed for these tests as they were included in the weight, but they offered little difficulty. As a result these tests have not demonstrated any new facts of importance regarding fitting design. The only recommendation on this point is that fittings should be carefully designed so no high stresses will be produced in carrying the applied load to both chords and the web, and that a little extra weight in them will usually increase the strength of the whole structure more than its weight.

DURALUMIN CHANNEL TRUSSES

134. Eleven duralumin and two steel channel trusses were tested. One of type 15, 8 of type 16, 2 of type 22, and 2 of type 30. On the whole it proved to be an excellent type, and though the strength-weight ratios obtained were not as great as the best from the box spars, the two types are very nearly equal in merit.

135. During the same period as that in which the spar tests were being made, Mr. R. A. Miller was carrying on an investigation of duralumin channels as columns on which he wrote Air Corps Information Circular, Vol. VI, No. 598 "Compressive Strength of Duralumin Channels." In this report he developed a method of computing the allowable stress in a channel used as a column and showed how his results could be used in the design of channel trusses. For further details the reader is referred to Mr. Miller's report.

136. As Mr. Miller's work is available and pertinent, it is not necessary to use a single arbitrary value for the allowable stress in the chord members of channel truss spars, as was done for boxes. When the channels are of the type covered by Mr. Miller's report they should be considered as pin ended columns of a length equal to the distance between truss joints and the allowable unit stress determined from the charts of Air Corps Information Circular, Vol. VI, No. 598. If Mr. Miller's charts are not applicable, the allowable stress must be determined with judgment extrapolating from Mr. Miller's charts if possible, but in no case assuming a yield point greater than 30,000 pounds per square inch. If possible, the yield point for the channel shape in question should be checked by tests of the type made by Mr. Miller in his study, in which case the value resulting from the tests may be used.

137. As the unit stresses obtained by Mr. Miller were from axially loaded columns, when used in spar design they should be considered as the average stresses in the channels. Therefore the designer of a channel truss should use the formula $f=P/A+M/A_ch$ instead of $f=P/A+M_c/I$ in computing the stress in the channel chords.

138. As in the case of the duralumin boxes, it is necessary to apply a stiffness factor to the value of E_sI_{gx} to compute the value of EI to be used in computing moments. A study is now in progress which aims at the development of a method of computing this quantity from a study of the deflection of the spar under the primary bending alone, but until this method has been perfected and checked by test, it will be necessary to use an arbitrary value. The ratios of EI_{xx}/E_sI_{gx} for the 13 channel trusses tested varied from 0.764 to 0.912, the median value being 0.826 and the mean 0.834. If the two steel trusses be neglected the range of variation remains unchanged but the median value becomes 0.815 and the mean 0.824. The ratios of EI_{mt} to E_sI_{gx} vary from 0.750 to 1.052, the median being 0.898 and the mean 0.899. If the steel spars are neglected the range of variation remains the same but the median value becomes 0.905 and the mean 0.912.

139. The values of EI_{xx} point to the use of 0.8 as the stiffness factor, while the EI_{mt} values point to 0.9. Mr. Miller has proposed in Air Corps Information Circular Vol. VI, No. 598 to use 0.9, but the writer prefers the more conservative value of 0.8, as he considers the values of EI_{xx} more reliable than those of EI_{mt}. In practice, it is not believed that the difference between these values is very great as the maximum stress in the beam is not very sensitive to changes in EI unless the value of L/j is large, and in that case it would be much better to use the more conservative figure, at least until a more rational method of computing the effective EI has been devised.

140. The margins of safety of the 11 duralumin channel truss spars were computed by the method recommended for future design, the computations being given in Table 14. The allowable unit stresses were taken from Air Corps Information Circular Vol. VI, No. 598, the remain der of the work being similar to that in Table 12. The most important changes in procedure were that the first trial axial load was taken in every case as the observed load at failure, and the formula for the average stress was used instead of that for the maximum stress.

TABLE 14.—*Computed strength channel truss spars*

FIRST TRIAL

	SPAR										
	15A	16A	16B	16C	16D	16E	16F	16G	16H	22A	22B
$E_s I_{gx}$	92.1	99.3	102.7	109.0	87.6	82.3	102.1	107.7	110.3	90.1	100.1
$EI=0.8\ E_s I_{gx}$	73.68	79.44	82.16	87.20	70.08	65.84	81.68	86.16	88.24	72.08	80.08
Allowable f	31,660	29,820	28,410	29,250	32,520	33,000	23,440	28,970	24,240	30,755	29,940
P_o	19,290	18,500	18,050	18,750	14,850	14,960	16,740	20,150	19,900	18,450	19,800
j	61.8	65.5	67.4	68.2	68.6	66.3	69.8	65.4	66.5	62.5	63.6
δ	0.825	0.718	0.670	0.650	0.639	0.695	0.613	0.718	0.690	0.803	0.768
M	59,300	54,900	52,700	54,400	42,890	44,090	47,920	59,800	58,500	56,370	59,730
A	1.160	1.261	1.247	1.350	1.173	1.166	1.188	1.355	1.386	1.127	1.219
A_e	3.220	3.505	3.552	3.800	4.059	4.326	3.473	3.785	3.878	3.939	3.817
P/A	16,630	14,660	14,480	13,890	12,650	12,830	14,090	14,870	14,350	16,370	16,240
M/A_c	18,410	15,660	14,840	14,320	10,560	10,190	13,800	15,800	15,085	14,300	15,650
f	35,040	30,320	29,320	28,210	23,210	23,020	27,890	30,670	29,435	30,670	31,890
SECOND TRIAL											
P	17,420	18,200	17,490	19,440	20,800	21,430	14,080	19,040	19,760	18,510	18,590
j	65.0	66.0	68.5	66.9	58.0	55.4	76.2	67.2	66.8	62.4	65.6
δ	0.729	0.703	0.642	0.681	0.990	1.130	0.493	0.673	0.682	0.807	0.715
M	51,900	53,700	50,600	57,000	67,400	72,500	38,630	55,700	57,900	56,600	55,200
P/A	15,010	14,430	14,020	14,400	17,730	18,380	11,850	14,050	14,250	16,430	15,260
M/A_c	16,100	15,320	14,240	15,000	16,600	16,750	11,110	14,700	14,930	14,370	14,450
f	31,110	29,750	28,260	29,400	34,330	39,130	22,960	28,750	29,180	30,800	29,710
THIRD TRIAL											
P	17,710	18,250	17,590	19,330	19,700	20,130	14,380	19,180	19,800	18,480	18,730
M. S. (per cent)	+8.9	+1.4	+2.6	−3.0	−24.6	−25.7	+16.4	+5.1	+0.6	−0.2	+5.7

Min. M. S. = −25.7%. Max. M. S. = +16.4%. Av. M. S. = −1.16%. Av. deviation = −9.05%.
Omitting 16D, 16E, and 16F. Min. M. S. = −3.0%. Max. M. S. = +8.9%. Av. M. S. = +2.64%. Av. deviation = 2.95%.
Corrected values of 16D and 16E −10.8% and −9.2%.

141. It will be noticed that the computed margins of safety for all of the spars in the group except 16D and 16E were small, and generally positive. 16F gave quite a high margin but it had very shallow chord members and a very low allowable stress. In this case it is believed that the webs and riveted joints appreciably decreased the slenderness ratio, so the allowable unit stress credited to this spar should have been higher, which would have reduced the margin of safety. On the whole this is not a serious defect in the recommended design rules, as it shows that when the designer is tempted to use a shallow channel, the unit stress will automatically become rather low, and he will be encouraged to make a more reliable design.

142. The large negative margins of spars 16D and 16E would be considered as pointing to serious defects in the proposed design rules, but after the spars had been tested, tests on the material used showed it to be of very poor quality. The computed strength of these spars was recalculated, reducing the allowable unit stress in the same proportion as the tensile strength of the material fell below 55,000 pounds per square inch. The revised margins of safety were −10.8 per cent for 16D and −9.2 per cent for 16E. These margins are about as great as is allowable, but it is considered that on account of the very poor quality of the material, these spars can be eliminated from consideration when formulating a rule for design. It is quite probable that the E of the material in these spars was low as well as the tensile strength, in which case a properly corrected pair of margin of safety values would be much closer to zero than those given above.

143. No discussion of the other questions arising in the design of duralumin channel trusses will be attempted in this report, as the subject is covered as completely as is possible in our present state of knowledge by Mr. Miller's Air Corps Information Circular, Vol. VI, No. 598. It may be well, however, to warn the reader that at least one cause of the discrepancy between Mr. Miller's ratios between the stresses carried by the spar chords and those given in this report is that he has given the stresses computed from the observed deflections, while the stresses in Table 14 are computed from an arbitrarily derived value of EI.

EFFECTS OF DESIGN FEATURES

144. The large number of design features incorporated in the relatively small number of spars tested precludes the possibility of making many general statements regarding the effects of particular features and demonstrating the truth of these generalizations mathematically from the results of the tests that have been made. It is possible, however, for one who has seen the actual tests and made the mathematical study of them recorded above to obtain a pretty good idea of the effects of the more important features of design, and the following paragraphs will cover the conclusions along this line at which the writer has arrived.

145. The first question to be taken up is the relative merits of trusses, boxes, and beams. All of the spars tested fall under one of these classifications, with the possible exception of those of type 18 which has characteristics of both the beam and the box, and type 11 which is intermediate between the truss and box

classes. On the whole the beams tested in this study have been unsatisfactory, due largely to their tendency to buckle laterally under relatively low loads. It is believed that this is due to the use of a single thin sheet at the plane of symmetry for the web, so that the web provides a minimum of resistance to the compression chord when it starts to bend out to the side. The reinforced web of 23A and the corrugated web of 23B showed up a little better in this respect than the webs of types 12, 13, and 14. This tendency of the beam to buckle laterally is its most serious defect, and it is believed that if it could be overcome, it would be the preferred type on account of its possible simplicity, accessibility, and adaptability to quantity production.

146. There seems to be little to choose between the better designs of boxes and trusses, and they seem to act in a very similar manner. None of these types showed much tendency to buckle under low loads unless they were very narrow. Under the loading used, it was essential however for the trusses to have webs approximately as wide as the chords, as in the cases of types 16 and 22 which had as web members channels extending the full width of the truss. The trusses with narrow webs, like types 17 and 31, seemed to have the same tendency to buckle laterally shown by the beams. It would seem that although there is little difference in merit between the best trusses and the best boxes under the loading used in this test, if the intensity of the loading were increased or the allowable spar depth decreased, the boxes would show up to greater and greater advantage. Conversely, if the intensity of loading were reduced or the allowable depth increased, the trusses would soon show to a definite advantage.

147. One great advantage of the beam and the box under intense loads and small spar depths is that the material in the web carries its full share of the axial load. Also, by carrying the shear by shear deformation, instead of by tension and compression in separate members, the deflections and secondary bending moments are decreased. It is probable, also, that the continuity of the web results in a somewhat higher modulus of rupture for the beam or box than can be obtained for the truss. In the greater depths encountered in the spars of bombers, for instance, these advantages are off-set by the great weight of the required webs, and it is more economical to use a truss type of web than to try to get equal results by means of lightening holes. Such holes in general give the spar the disadvantages of both the box and truss types. The tests on the spars of type 10 seem to point in the opposite direction, but the writer believes that the actual effect was that the flanges to the lightening holes in spar 10E gave more support to the cover plates used than the flat plate webs of 10C and 10D. If, however, these cover plates had been made of the proper size, it is thought that the spars without lightening holes would have shown up better than those with them. This is indicated by the results on spars 10A and 10B.

148. The best ratio of spar width to spar depth has not been indicated by the tests made to date. It seems, however, that the wider a spar can be made without danger of spreading the material in the chords out so thin that it will buckle locally, the greater will be the EI_{yy} of the spar, and the less the danger of lateral buckling of the compression chord. The minimum width of the spar is not a quantity that can be definitely determined with our present knowledge, but the designer should try to obtain as high a value of EI_{yy} as he can. It is hoped that further research will indicate a method of determining the needed magnitude of this property, as the narrower the spar with an adequate EI_{yy} the more efficient it can be made for the resistance to loads in its plane. This problem of the needed width of a spar is complicated by the possible effect of the ribs in providing the spar with the lateral support needed, a phase of the general question not treated in this report.

149. The further the centroids of the chords of a spar are from the neutral axis, the lower will be the unit stresses required to resist a given bending moment and the smaller will be the deflection under a given load and the resulting secondary bending moments. If this criterion could be followed without qualification, the best type of chord member would be a flat plate such as was used in type 10. The weakness of this type of chord member is that, considered by itself, it has a very small radius of gyration about an axis through its centroid parallel to the minor axis of the spar, and will buckle locally under a relatively small load. In trusses, this weakness is countered by using channels or tubes for the chord members, the properties of these members about their own major axis being made comparable to those of the whole spar about its major axis. In the spars of type 10, it was hoped that the webs riveted to the cover plates along their edges would provide the needed stiffness. The tests showed, however, that this was not enough, as rivet heads would be pulled off suddenly doubling the unsupported length of the chord member and it would then fail. A better method of obtaining the desired result was by using chord members like those of types 20 and 21 which did not have to depend so largely on rivets for their stiffness.

TABLE 15.—*Standing of types*

Spar	P/Wt.	Per cent of 1658	Spar	P/Wt. ×%D. L.	Per cent of 1658	Spar	$\frac{P/Wt.}{\sqrt{\% Des. P}}$	Per cent of 1,658
2C	1,658	100.0	2C	1,658	100.0	[1] 17B	1,732	104.5
20B	1,615	97.4	20B	1,615	97.4	2C	1,569	94.6
4C	1,603	96.7	4C	1,603	96.7	4C	1,554	93.8
[1] 17A	1,555	93.8	[1] 30A	1,518	91.6	16A	1,515	91.4
[1] 30A	1,518	91.6	21D	1,499	90.4	[1] 23B-2	1,514	91.3
21D	1,499	90.4	16G	1,497	90.3	[1] 30B	1,509	91.0
16G	1,497	90.3	22B	1,444	87.1	23B-1	1,506	90.8
22B	1,459	88.0	10B	1,383	83.4	20B	1,490	89.9
15A	1,419	85.6	15A	1,367	82.4	22B	1,466	88.4
18B	1,401	84.5	[2] 14C	1,363	82.2	[1] 11B	1,454	87.8
10B	1,383	83.4	[1] 32B	1,342	81.0	18A	1,446	87.3
[1] 32B	1,365	82.3	18B	1,335	80.6	15A	1,445	87.2
[2] 14C	1,363	82.2	1A	1,265	76.3	10E	1,443	87.1
[1] 14B	1,361	82.1	[1] 14B	1,259	76.0	21B	1,438	86.8
23A	1,359	82.0	[1] 17A	1,257	75.8	[1] 14B	1,415	85.4
[1] 23B-2	1,344	81.1	14D	1,225	73.9	[1] 32B	1,377	83.1
[1] 11B	1,310	79.0	23A	1,195	72.1	11A	1,328	80.1
1A	1,265	76.3	31C	1,194	72.0	[1] 14C	1,296	78.2
14D	1,245	75.1	[1] 12B	1,115	67.2	14A	1,275	76.9
[1] 12B	1,202	72.5	[1] 11B	1,065	64.2	1A	1,245	75.1
31C	1,194	72.0	[1] 23B-2	1,059	63.8	[1] 12B	1,225	73.9
11A	1,182	71.3	11A	938	56.6	[1] 13B	1,208	72.9
[1] 13B	1,095	66.0	40B	920	55.5	13A	1,168	70.5
40B	1,005	60.6	[1] 13B	900	54.3	31C	1,136	68.6
12A	999	60.3	12A	794	47.9	12A	1,121	67.6
13A	992	59.7	13A	712	42.9	40B	1,051	63.4

[1] Indicate spars tested with two lateral supports.

[2] Spar tested with four lateral supports.

150. Table 15 gives the relative ratings of the different types of spars in the first two groups of Table 4 in accordance with the strength-weight ratio and its modifications shown in that table. Only one spar is listed for each type, except when some of the spars of a type were tested under design conditions, and others with additional lateral support. In every case it is the best spar of each group for which the figures are recorded, and the letter indicating the spar to which the data pertains is shown.

151. From the data of Table 15 it can be seen that the best type of spar tested was the conventional wood box, if we consider the actual strength-weight ratio developed to be the measure of merit. An interesting phase of this matter is that all of the wood spars of types 1, 2, and 4 were designed and tested at the beginning of the study, while most of the metal spars which showed up well, did so only after the types in question had been developed after a series of trials. This fact, indeed, summarizes the present relative status of wood and metal for use in spar design. So much experimentation has been done in the past on the design of wood spars that it is possible to design wood boxes and I beams to carry a given load and get a spar that will carry the desired load, or very close to it, with a good strength-weight ratio on the first attempt. In the case of metal, the designer is working in a relatively new field in which the best types of design have not been decided upon, and the allowable unit stresses for the different types of construction and relative dimensions have not been determined. The result is that several trials are usually needed before a satisfactory design is developed. In fact, some of the metal spars which showed up best on their first trials were designed by following wood design practice as closely as possible. At present this lack of knowledge and experience with metal design is the chief reason for the preference of many engineers for wood construction, and the fundamental purpose of this study is to rectify it so far as possible and determine which types of metal design show the most promise of producing spars of equal merit to the conventional wood types, and to study the factors that must be taken into consideration in their design.

152. On the whole, the strength-weight ratio figures show that there are two types of metal spars that promise to equal the wood spar in merit. These are the duralumin box, as represented by types 20, 21, and 10, and the duralumin channel truss as represented by types 16, 22, and 15. At present, the boxes show up a little better than the channel trusses, both types 20 and 21 showing higher strength-weight ratios than type 16, one of which showed up better than any of the other channel trusses. The spars of types 20 and 21 which showed up so well were among those most recently tested, and before the tests on them, the channel trusses seemed to be somewhat better than the boxes. It is quite possible that further development of the channel trusses will again bring them up to and beyond the figures for the boxes. Considering the fundamental properties of these two types, it seems that the loading used in this test, including the limitation as to spar depth, is very close to that at which the two types are of equal merit. If the loading were more intense or the allowable depth less, it is believed that the superiority of the boxes over the channel trusses would become much more pronounced. On the other hand, if the loading were less intense, or the allowable depth greater, it is believed that the channel truss would show a pronounced superiority over the box.

153. It might be claimed that spar 20B showed an inferior strength-weight ratio to spars 19A and 19B and that type 19 should be considered the most promising substitute for the wood types. These high ratios of the type 19 spars, however, were due primarily to their eccentric loading and camber. That the same results could be obtained with any type of spar, particularly a wood box, is shown by the results of the tests of the type 3 spars, one of which gave a higher strength-weight ratio than even 19B, although, unless

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and their supposed causes, when rivets began to fail, and any such events. Sometimes one observer will think that the web or the compression chord is buckling but the others will disagree. In such cases this fourth man should enter these facts in the log. In fact he should try to note all remarks made by the witnesses as to the action of the spar under load that are pertinent to the test. After the spar has failed he should write a complete description of both primary and secondary failure. As soon as possible after the test he should write a short report on the test from his rough log and file it with log of the test kept by the third man. In this fourth man's log the events can be dated by the deflection readings as they are easier to keep track of than the loads, but in his later report they should be translated into the corresponding loads taken from the log of the third man.

173. Tests should be carried to destruction with sufficient permanent set to show the character of the failure whenever possible. An exception to this rule may be made for spars that fail by lateral buckling of the compression chord, but even in these cases it is desirable to stop the test only when some part of the compression chord has crushed locally. After several spars have been tested, a picture should be taken of the group, showing the failures as clearly as possible. In some cases, a picture showing the whole spar will not show the failure very clearly, and while such spars should not be omitted from the group pictures showing the entire lengths of the spars, additional "close-ups" should be taken of the portion that failed. By following the suggestions outlined above, it is believed that the test records will be found much more reliable than those made in the past, and proportionally more useful.

STAGES OF STUDY

174. The study of the different types of spar design may be divided into three main stages which will be referred to below as prospecting, development, and intensive study. The prospecting stage is that at the beginning of the study when it is not at all certain whether or not the type will be found efficient. Most of the study recorded in this report was on types in the prospecting stage. As a result of this prospecting stage, two types, the duralumin box and the duralumin channel truss, were selected as worthy of further study and have been put into the development stage. This implies that the general lines of those types showed much promise and the study of them was continued to make sure that they were good. Several other types have also been selected for development such as the extruded duralumin I beam, and the alloy-steel channel truss. The third stage, intensive study, is that in which as many spars as possible are tested in order to determine the exact effect of the various design variables and to formulate satisfactory rules for design. This stage has been entered for the duralumin box and channel-truss types. Thus it can be seen that while there is a very definite distinction between the prospecting and intensive study stage, the development stage has vague boundaries and shades off into the other two without distinct lines of demarcation.

175. In the case of types still in the prospecting stage the division should leave the design entirely to the proposer of the type, giving him a free hand. In the development stage, the designer should be required to submit figures to the division to create a presumption that the changes proposed by him will probably result in improved performance, though in some cases, the division will ask the designer to incorporate certain features. In the intensive study stage the division should specify the critical dimensions of the spars desired, though in practice the planning of a series of designs will usually be made in cooperation with a contractor. It should not be necessary for the division to make complete detailed drawings for the spars it purchases in the intensive study stage, but sketches showing the critical dimensions desired should be sufficient even when the spars are advertised for competitive bids. Certainly this should be the case when the spar is covered by design rights not yet purchased by the division and the one constructor is the sole source of supply.

SPAR TYPES TO BE STUDIED

176. In discussing the types of spars to be considered in further work, distinction must be made between spars of the size used in the current study and other sizes, and also between the three stages of study as defined above. The work that should be carried out on the observation size used in the current study will be discussed first and then some remarks will be made regarding tests on other sizes.

177. The tests made to date have indicated that the most promising types are the duralumin box and the duralumin channel truss. Intensive study has been started on both of these types, resulting in the formulation of the design rules given earlier in this report, but the discussion of those rules shows how numerous are the questions remaining to be solved before rules for their design can be formulated to cover all probable cases. In the intensive study proposed for these types, particular attention should be paid to the hexagonal cell box-like type 20, as the proposed design rules are too conservative for this type which showed the highest strength-weight ratio of all of the metal spars tested under the specified conditions. Parallel to the intensive study of these types there should be a further development study of the extruded I, plate girder, hourglass, and dumb-bell types in duralumin and the heat-treated steel channel truss. While carrying on the intensive study and development of the types mentioned, a small amount of prospecting should also be done if any types are proposed that look promising, but the emphasis should now be put more on development and intensive study than on additional prospecting. It is not considered that further attention need be paid to the welded steel tube truss as represented by types 31 and 32, the combination steel and duralumin tube truss as represented by type 40, or the spar with round duralumin tube chord members, as represented by type 11.

178. Although two spars of both types 17 and 23 were tested, all four of them were original designs

without the advantage of a preliminary trial, as in both cases the two spars were designed and built at the same time, the chord members being practically the same but the webs being of different character. This was done in the hope of getting information regarding the relative merits of the types of web used, but the results were not satisfactory as, in all four cases, failure was by lateral buckling of the compression chord, and no difference in action due to the differences in web design could be detected.

179. The spars of type 17 showed up very well, the best of the pair being exceeded in strength-weight ratio by only the wood boxes, one of the wood I beams, and the dural box 20B. The other spar of type 17 was also bettered by spar 30A. The main reasons why this type has not been studied more intensively are that it lacks lateral stiffness, that the high strength-weight ratios obtained were developed only in tests with two points of lateral support instead of only one, and the fact that the load at failure was only about 0.8, the design load. The low strength of the spars tested is not a valid reason for losing interest in the development of the type, though it would prevent its being used until more reliable methods of design could be developed. In fact the combination of low strength and high strength-weight ratio would indicate that as a type the dumb-bell was worthy of serious development. It seems almost certain that it should be practicable to so increase the lateral moment of inertia of spars of this type as to provide adequate lateral stiffness, and at the same time increase the strength to the desired figure without increasing the weight sufficiently to decrease the strength-weight ratio. The only feature of the construction that is of doubtful merit is the lack of torsional strength due to the use of a single web in the plane of symmetry of the section. None of the spars that had single webs in the plane of symmetry showed up well except the wood I, and in this case the web was much thicker than for the metal designs. Whether torsional strength is really needed in a spar that is held in place by drag struts, ribs, and the other members of the conventional cellule, is open to question, but it seems obvious that a type of construction that has inherent torsional strength is better than one which has equal merit otherwise, but lacks this one quality. A practical disadvantage of the dumb-bell type is the necessity of using a chord of irregular section, the geometrical properties of which are difficult to determine. This makes it hard to design a spar with given properties, and thus makes the type less acceptable to the designer.

180. The dumb-bells of type 23 did not show up nearly as well as those of type 17. This may have been due to the fact that both spars were tested with single points of lateral support. It is true that spar 23B was tested also with two points of support, but the test was made after the test with a single support, and that fact that it carried little more load in the second test than in the first may have been due to its having been injured in the first test. It is also possible that the shape of the chord member used in type 23 was inferior to that used in type 17, or the use of a continuous sheet web instead of light trussing may have been less economical.

181. Type 30 was very similar to types 16 and 22, except that the material used was heat-treated steel instead of duralumin. Owing to the greater density of the material, it was obvious that local failure would result if the chord channels were made as wide as those of type 16. When the spars had been constructed and were tested in simple bending, it was noted that the lateral EI value was low, due to the narrowness of the spars. This indicated that the spars would fail at a very low load if tested with a single lateral support, so two were used. One of the two spars was connected by rivets, and the other by welding after heat treatment. The welded spar showed that the properties of the material had been lowered by the welding, and it failed at a lower load and strength-weight ratio than the riveted spar, in spite of the fact that the connections were lighter.

182. The studies of duralumin channels of Air Corps Information Circular, Vol. VI, No. 598, show that if a steel truss spar of this character were made wide enough to need only one lateral support, either the amount of material in the chords would have to be increased so much as to lower the strength-weight ratio excessively, or failure would occur at a low load due to local buckling. It would be advisable, however, to try a spar of this general character but with the back of the chord channel reinforced by a longitudinal corrugation like those of spars 16G and 16H.

183. It might also be desirable to try the type again with two lateral supports to see if it can not be improved for use when there will be adequate lateral restraint provided externally. Before the latter is done spars of the dural box and dural channel truss types should be designed and tested with two lateral supports, as it is believed that the strength-weight ratios of these types could be improved if they did not have to be made so wide. If this should turn out to be the case, it would indicate that they were superior to the types that required the additional support before they will show up favorably.

184. The duralumin hourglass of type 18 is a hybrid between the box and dumb-bell types. The second one tested showed up quite well, and its designer believes that by slight modifications it can be greatly improved. This should be checked by tests, and if it should show up as well as its designer hopes it will deserve intensive study. Until then it should be considered only as a type that should continue under investigation but not intensively.

185. The I beams of duralumin did not show up well. In the cases of the extruded beams of types 12 and 13 this was due in part, at least, to the fact that manufacturing conditions limited them to a depth of 5¾ inches, whereas the other spars tested could be made 6¼ inches in depth. This difference in depth of one-half inch in six was too great a handicap for them. It must be admitted, though, that the spars of type 14 in which the full depth was utilized did not show up very much better. If the type were one of no extensive use in conventional structural use, like type 17, for example, it would probably be said that the fundamental cause of weakness was the lack of torsional stiffness and strength inherent in the use of a single web in the plane of symmetry, and study of the type would be

dropped. The I beam and the plate girder, however, are among the most widely used types of construction in conventional steelwork, and it should not be dropped from airplane work on the basis of the few tests made on them in this study. It appears to the writer that on this account further attempts should be made to develop this type, as he does not believe that those tested represented the best available design. The designer of types 12 and 13 claims that it is now possible to obtain extruded I beams up to the 6¼-inch depth, and it would be highly desirable to test out a spar of type 12 in this depth. If it or a new spar of type 14 showed up well, the type could then be subjected to more intensive study.

186. Even though the extruded I-beam or the plate girder should not show up as well as some other types, if it showed up reasonably well, it would have to be considered a standard type and the constants for its design would have to be determined. For quantity production of constant chord airplanes, the extruded I is by far the simplest possible type of spar, and might often allow the construction of a lighter wing than could be obtained with more complicated structures that showed up better on strength-weight ratio in tests. The plate girder would not be so good for quantity production, but for small orders and for tapered wings, if a few sizes of extruded angles or tees were available, it would be possible to design very satisfactory spars at reasonable cost. Often it would be possible to make the original design with plate girder spars, and then if the design was satisfactory and a production order large enough were desired, an extruded I of the same properties could be substituted with a resulting saving in weight. Type 13 does not warrant further study, as it showed up in the tests as inferior to type 12.

187. The tube chord designs like types 11, 31, 32, and 40 have not shown up with sufficient merit to warrant their further study. If the tubes are made round, the effective depth of the spar is too much reduced to permit an efficient design, as is shown for duralumin by the high unit stresses and low strength weight ratios of type 11. Even though steel is used, the effective depth would be less than for a design with elliptical tubes. In addition, the use of round tubes involves an unnecessary sacrifice of lateral stiffness. With elliptical tubes, these types look more promising, but the comparative figures indicate that the material can not be as efficiently located when tubes are used as chord members as when the channels of the channel truss type, or the flat plates or shallow channels of the box type are employed. This is particularly serious when duralumin tubes are used, as the fitting design becomes a difficult problem, it being possible to make good connections between the flat plates of the box and channel truss types much more easily and efficiently than between tubes. With steel, the difficulty can be reduced by the use of welding, but this prevents the use of the high heat-treatments needed to get the efficiency required for acceptable design. The spars of type 32 and spar 30B were welded after heat treatment. Table 4 shows that the riveted spar 30A is much superior to the other three, and the difference is almost certainly due to the welding in the latter cases.

TESTS ON OTHER SIZES

188. The tests made to date, though they have not satisfied our curiosity regarding the design of spars suitable for observation airplanes, have given us much more knowledge regarding which types are efficient in this size and how to design them than we have regarding spars of either larger or smaller sizes. Therefore while further study of the observation size should not be dropped, study should be initiated of the design of metal spars of other sizes. The problems involved are primarily the determination of the size to be studied and the working out of a satisfactory test rig for the size selected.

189. In pursuit and training airplanes, the allowable depth of spar is less than 6 inches in many cases, and some study is needed on spars of this class. Whether this class is more or less important than the deeper spars will not be discussed at length in this report, but it seems obvious that sooner or later, it will be necessary to study metal spars of approximately 4-inch depth, as designers will insist on using metal spars in the smaller sizes of airplane, and the writer believes that metal is more likely to prove superior to wood in these shallow spars than in any other size.

190. When the observation loading used in the tests discussed in this report was worked out, other loadings were suggested in Serial Report 2450, "Loadings for Experimental Airplane Spars," by J. S. Newell, for both larger and smaller spars, but none of them have been used in actual tests. An attempt was made to design spars for the pursuit loading of that series, but it was found that the resulting secondary bending moments were excessive. More recently, a new pursuit loading was worked out and its suitability checked by making trial designs and comparing the design loads including secondary effects with the same quantities for actual pursuit airplanes. The comparison was satisfactory and the resulting figures for the loads and limiting dimensions are given in the requirements for experimental metal spars which are proposed for use in making future purchases. The test jig for this loading has not yet been constructed and no tests have been made so, although it is believed that this loading will prove satisfactory, this can not be proved until it has been tried out. The bomber loadings proposed in Serial Report 2450 are not believed sufficiently severe, nor the limiting dimensions sufficiently large for use in future work. Before tests are made on spars intended to be suitable for large bombers, a new loading should be worked out based on the critical loads on the spars of the XHB-1, LB-5, and similar airplanes.

191. Several years ago a series of tests were made on metal spars 15 inches deep and of a size suitable for use in bombardment airplanes. The results of those tests are recorded in Air Corps Information Circular No. 556, "Comparison of Tests on Experimental 15-Inch Metal Spars and 11-Foot Chord Metal Wing Ribs," by J. S. Newell. These tests showed the welded chrome-molybdenum steel tube truss similar to types 31 and 32 to be the best of the metal types tested and about equal in merit to the wood box. In fact, it was this series of tests which caused the

popularity of the welded tube type of truss for large airplane spars.

192. It might be urged that in future tests the same loading and allowable depth should be used, but this is not considered desirable. In the tests of 15-inch spars it required nearly a day to set up a spar ready for test and about half a day more to test it. This time element has been greatly reduced for the 6¼-inch spars by the use of a jig in a testing machine, and early in 1927 17 spar tests were made between January 26 and February 2, inclusive, not less than 2 tests being made on any 1 day, 3 tests being made on each of 3 days, and on 1 day 4 tests. Two of the days in the period were a Saturday and a Sunday. It would have been absolutely impossible to do so much testing with the loading of Air Corps Information Circular No. 556.

193. A more important defect of the loading of Air Corps Information Circular No. 556 was that the load had to be applied in increments, usually of 5 or 10 per cent of the design load, and the load at failure could not be determined closer than to the nearest increment. In the testing machine, the axial load at failure could be read to the nearest 10 pounds, but it must be admitted that the probable error was nearer 20 or 30 pounds. But even if the error had been 100 pounds, the result would have been much more precise than the nearest 5 per cent increment.

194. In the tests of Air Corps Information Circular No. 556, when the maximum load was reached the spar collapsed and it was never possible to get points representing conditions after the maximum load had been passed, as was possible with several of the 6¼-inch spars. This made it harder to determine which were primary and which secondary failures. The loading was also imperfect in that the maximum bending moment came at the fitting at which the axial load was applied and it was usually difficult to determine whether the failure should be charged against the fitting or whether the spar itself had also been subjected to the maximum load it could carry.

195. All of the above defects in the method of testing were eliminated in the case of the 6¼-inch spars by the new method of testing, and it is considered that to return to the old method would be a retrogression that should be avoided if at all possible. So far, however, it has not been possible to devise a method of testing the deeper spars that appeared satisfactory. It would not do to simply increase the allowable depth and modify the axial and side loads required for the 6¼-inch spars, as the ratio of length to depth of a 12-inch spar would be only 8, and this is considered too small to be satisfactory. It is also impracticable to increase the length unless some means is devised to increase the length of specimen that can be handled with the available testing machines.

196. The problem is further complicated by the uncertainty as to the trend of design and therefore as to the character of the loading that should be used. One strong tendency is toward the tapered cantilever monoplane, and another is to the externally braced biplane. The spar depth of the first type is subject to very great variations, but on the whole it presents the easier problem, as the spar is required to carry only simple bending. Two or three spar depths covering the probable range could be selected and tests made in all of these depths. The tests could be made in the available testing machines, the spars being subjected to two concentrated loads near the center of a simple span, the reactions being provided by supports from a heavy steel I beam below the test spar. Loadings, and means of providing lateral support to the compression chord, and for reading deflections would have to be worked out, but that would not be a difficult problem. If a system were developed for testing deep spars under combined axial and bending loads, tests of this character would probably be desired in any event, as they are provided for as part of the preliminary bending tests mentioned in the new requirements for 6¼-inch spars.

197. Preliminary studies of the problem have indicated that it will be practicable to design jigs in which to test spars of greater length than eight feet in the available testing machines. As the roof of the new laboratory building at Wright Field is much higher than that in the old material branch building at McCook, it may be possible to fit longer extension screws on the testing machine used in the majority of the tests recorded in this report and test the spars in a jig of the same type as that used on the 6¼-inch spars. Even if this should not prove practicable, methods can be devised to handle the larger spars, two such methods having been sketched out. Both of these new methods, however, require more complicated and expensive jigs than the type used on the observation size of test spars, and considerable development and design work would still be needed before they could be constructed and ready for use. It is recommended, therefore, that the possibility of increasing the length of the specimen that can be tested with the type of jig now used be further investigated, and if it is not possible to do so to the extent necessary to handle test spars of a suitable length for the bomber size, the alternative methods of testing be given further study and a jig constructed for testing by what is decided upon as the better method. Whatever method of test is decided upon, however, no metal spars should be purchased for test until the suitability of the method of test selected has been proved by tests on wood box spars constructed by the division.

198. The decision as to whether the next size to be studied should be larger or smaller than the 6¼-inch depth should depend upon the type of airplane in which the division will be most interested two or three years from now, as it will probably be that long before the relative merits of the different types of construction can be determined. If funds are available, it would be desirable to carry out tests on both pursuit and bomber sizes. It is believed that results will be obtained more quickly in the additional sizes than in the past with the 6¼-inch sizes, as many of the results obtained with one size will be found applicable to the others, and the tests of one size will act as a guide to the tests needed in the others. Of the three sizes, it is recommended that the observation be the one in which the greater part of the study be made as its loads and limiting dimensions are intermediate between those of the other two.

APPENDIX I

LOGS OF TESTS 6¼ INCH METAL SPARS

Logs of tests

Test 10A-1		Test 10A-2		Test 10B	
Load	Deflection	Load	Deflection	Load	Deflection
312	0.000	312	0.000	312	0.000
1,250	.027	1,250	.026	1,250	.033
2,500	.059	2,500	.060	2,500	.064
3,750	.092	3,750	.093	3,750	.094
5,000	.125	5,000	.132	5,000	.128
6,250	.160	6,250	.171	6,250	.162
7,500	.200	7,500	.211	7,500	.198
8,750	.243	8,750	.251	8,750	.238
10,000	.288	10,000	.292	10,000	.280
11,250	.338	11,250	.337	11,250	.327
12,500	.392	12,500	.383	12,500	.374
13,750	.446	13,750	.430	13,750	.428
14,375	.472	15,000	.480	15,000	.483
15,000	.501	16,250	.529	16,250	.546
16,250	.568	16,875	.554	16,875	.582
16,875	.606	17,500	.580	17,840	.650
17,500	.646	18,125	.607	18,310	.680
18,125	.638	18,750	.636	18,750	.720
18,750	.765	19,375	.685	19,180	.760
				19,430	.800
				20,125	.900
				20,185	.950
End of fitting hit lever arm of test jig. One latera support.		Failure of rivets in tension, 10 inches from center. One lateral support.		Tension failure of rivets, 10 inches from load point. One lateral support.	

Test 10C		Test 10D		Test 10E	
Load	Deflection	Load	Deflection	Load	Deflection
250	0.000	250	0.000	250	0.000
1,000	.025	1,000	.019	1,000	.024
2,000	.051	2,000	.046	2,000	.058
3,000	.077	3,000	.077	3,000	.090
4,000	.103	4,000	.103	4,000	.122
5,000	.134	5,000	.131	5,000	.155
6,000	.163	6,000	.159	6,000	.185
7,000	.194	7,000	.190	7,000	.223
8,000	.226	8,000	.220	8,000	.258
9,000	.259	9,000	.253	9,000	.293
10,000	.296	10,000	.287	10,000	.332
12,000	.375	12,000	.358	12,000	.413
13,000	.426	13,100	.400	12,900	.450
14,000	.487	14,220	.450	13,900	.500
14,900	.550	15,250	.500	14,740	.550
15,750	.650	16,500	.600	15,440	.600
15,880	.700	17,010	.650	16,120	.650
15,900	----------			16,700	.750
				16,790	----------
Rivets in compression flange failed 19 inches from side load. One lateral support.		Rivets in compression chord failed $10\frac{11}{16}$ inches from side load. One lateral support.		Rivets in compression chord failed 20 inches from side load. One lateral support.	

Logs of tests—Continued

Test 11A		Test 11B		Test 12A	
Load	Deflection	Load	Deflection	Load	Deflection
312	0.000	312	0.000	312	0.000
1,250	.018	1,250	.044	1,250	.008
1,875	.045	2,500	.117	2,500	.044
3,125	.109	3,750	.180	3,750	.082
4,375	.177	5,000	.240	5,000	.120
5,000	.213	6,250	.307	6,250	.158
6,250	.287	7,500	.378	7,500	.200
6,875	.326	8,750	.454	8,750	.243
8,125	.409	10,000	.537	10,000	.287
9,060	.471	11,250	.630	11,250	.334
10,000	.540	12,500	.735	12,500	.384
11,310	.640	13,125	.800	13,750	.437
12,000	.700	14,130	.920	15,000	.493
13,200	.820	15,000	1.100	15,875	.600
14,180	.940	15,570	1.250		
15,120	1.100	16,150	1.500		
15,710	1.130	16,250	1.650		
15,860	1.400	16,250	1.750		
Failure by lateral buckling resulting from bowing of lower half of spar. One lateral support.		Failure by lateral buckling between 2 side supports. Two lateral supports support lateral at $\frac{1}{3}$ points.		Failure by lateral buckling. One lateral support.	

Test 12B		Test 13A		Test 13B	
Load	Deflection	Load	Deflection	Load	Deflection
312	0.000	312	0.000	312	0.000
1,250	.030	1,250	.013	1,250	.027
2,500	.080	2,500	.061	2,500	.085
3,750	.115	3,750	.102	3,750	.127
5,000	.154	5,000	.147	5,000	.175
6,250	.193	6,250	.194	6,250	.222
7,500	.234	7,500	.242	7,500	.272
8,750	.274	8,750	.297	8,750	.322
10,000	.317	10,000	.350	10,000	.370
11,250	.362	11,250	.402	11,250	.424
12,500	.410	12,500	.457	12,500	.478
13,500	.450	13,125	.488	13,125	.515
14,630	.500	14,375	.580	14,375	.550
15,800	.550	13,750	.650	13,750	.585
16,800	.600			15,000	.625
17,750	.650			15,625	.678
19,050	.750			16,375	.780
19,200	.800			16,425	.800
19,240	----------				
Failure by lateral buckling. Two lateral supports at $\frac{1}{3}$ points.		Failure by lateral buckling. One lateral support.		Buckled in middle span. Failure by lateral buckling; buckled to S curve. Supported laterally at $\frac{1}{3}$ points.	

Logs of tests—Continued

Test 14A

Load	Deflection
312	0.000
1,250	.033
2,500	.054
3,750	.087
5,000	.119
6,250	.152
7,500	.188
8,750	.226
10,000	.264
11,250	.305
12,500	.352
13,125	.374
14,240	.420
15,000	.460
15,340	.480
15,500	.500

Web started to buckle.
Failure by lateral buckling.
One lateral support.

Test 14B

Load	Deflection
312	0.000
1,250	.028
2,500	.063
3,750	.098
5,000	.140
6,250	.175
7,500	.210
8,750	.247
10,000	.283
11,250	.323
12,500	.364
13,750	.412
14,550	.440
15,950	.500
16,820	.550
17,440	.600
18,500	.700

Web buckled near lower fitting.
Local failure at midspan.
Two lateral supports at ⅓ points.

Test 14C

Load	Deflection
250	0.000
1,000	.027
2,000	.058
3,000	.085
4,000	.110
5,000	.133
6,000	.162
7,000	.190
8,000	.220
9,000	.247
10,000	.273
11,000	.300
12,000	.327
13,000	.354
14,000	.382
15,000	.410
16,000	.444
17,650	.500
19,000	.550
21,000	.650
22,100	.800
21,750	.850

Failure by lateral buckling.
Four supports, each at a ⅕ point.

Test 14D-3

Load	Deflection
250	0.000
1,000	.017
2,000	.050
3,000	.075
4,000	.100
5,000	.126
6,000	.148
7,000	.172
8,000	.196
9,000	.220
10,000	.245
12,000	.298
13,920	.350
15,480	.400
16,850	.450
17,970	.500
18,900	.550
19,570	.600
19,690	.650
19,690	.700

Failure by lateral buckling.
One lateral support.

Test 15A

Load	Deflection
250	0.000
1,000	.027
2,000	.110
3,000	.140
4,000	.178
5,000	.205
6,000	.235
7,000	.273
8,000	.305
9,000	.340
10,000	.416
12,000	.496
14,000	.600
16,250	.700
17,800	.800
18,800	.900
19,240	1.000
19,290	1.100
17,740	----------

Lateral failure of compression flange.
One lateral support.

Test 16A

Load	Deflection
250	0.000
1,000	.022
2,000	.053
3,000	.084
4,000	.113
5,000	.144
6,000	.176
7,000	.210
8,000	.243
9,000	.276
10,000	.312
12,000	.384
14,000	.467
15,750	.550
16,500	.600
17,580	.700
18,100	.800
18,330	.850
18,460	.900
18,500	.950

Failure by crushing of compression chord.
One lateral support.

Logs of tests—Continued

Test 16B

Load	Deflection
100	0.000
1,000	.023
2,000	.052
3,000	.080
4,000	.106
5,000	.130
6,000	.155
7,000	.181
8,000	.203
9,000	.230
10,000	.257
12,000	.320
14,200	.400
16,500	.550
17,250	.650
17,790	.800
17,970	.900
18,030	.950
18,050	1.000
18,050	1.050

Failure by crushing of compression chord 3 inches from center.
One lateral support.

Test 16C

Load	Deflection
100	0.000
1,000	.034
2,000	.066
3,000	.095
4,000	.120
5,000	.150
6,000	.173
7,000	.200
8,000	.223
9,000	.250
10,000	.278
12,000	.337
13,900	.400
15,250	.450
16,870	.550
17,980	.700
18,500	.850
18,650	.950
18,720	1.000
18,740	1.050
18,750	1.100

Failure by crushing of compression chord 3 inches from center.
One lateral support.

Test 16D

Load	Deflection
100	0.000
1,000	.034
2,000	.073
3,000	.108
4,000	.139
5,000	.172
6,000	.202
7,000	.232
8,000	.265
9,000	.297
10,000	.332
12,000	.424
13,100	.500
14,050	.650
14,180	.700
14,460	.850
14,580	1.000
14,620	1.100
14,620	1.150
14,580	----------

Failure by bending of a section of the compression chord.
One lateral support.

Test 16E

Load	Deflection
100	0.000
1,000	.035
2,000	.077
3,000	.105
4,000	.138
5,000	.170
6,000	.203
7,000	.233
8,000	.268
9,000	.303
10,000	.342
12,000	.443
14,100	.650
14,450	.750
14,750	.900
14,900	1.050
14,950	.200
14,960	.250
14,960	.360
14,940	.350
14,920	.450
14,900	1.500

Failure by lateral buckling.
One lateral support.

Test 16F-1

Load	Deflection
100	0.000
1,000	.030
2,000	.077
3,000	.111
4,000	.143
5,000	.176
6,000	.207
7,000	.240
8,000	.273
9,000	.308
10,000	.342
12,000	.415
13,840	.500
15,330	.600
16,130	.700
16,396	.750
16,450	--------

Compression diagonals not reinforced. Second compression diagonal from each end failed.
One lateral support.

Test 16F-2

Load	Deflection
100	0.000
1,000	.031
2,000	.068
3,000	.100
4,000	.137
5,000	.173
6,000	.207
7,000	.243
8,000	.282
9,000	.321
10,000	.361
12,000	.442
13,400	.500
14,500	.550
15,600	.600
16,570	.650
16,740	----------

Compression diagonals reinforced. Failure by bending of compression flange 6 inches from midlength.
One lateral support.

Logs of tests—Continued

Test 16G		Test 16H		Test 17A	
Load	Deflection	Load	Deflection	Load	Deflection
100	0.000	100	0.000	250	0.000
1,000	.024	1,000	.034	1,000	.030
2,000	.060	2,000	.070	2,000	.073
3,000	.087	3,000	.098	3,000	.117
4,000	.116	4,000	.128	4,000	.165
5,000	.141	5,000	.157	5,000	.214
6,000	.170	6,000	.184	6,000	.257
7,000	.197	7,000	.212	7,000	.300
8,000	.225	8,000	.238	8,000	.350
9,000	.250	9,000	.265	9,000	.395
10,000	.281	10,000	.297	10,000	.446
12,000	.345	12,000	.362	12,000	.570
13,750	.400	13,180	.400	13,750	.700
15,000	.450	14,500	.450	14,900	.800
17,100	.550	16,680	.550	15,850	.900
18,900	.700	18,140	.650	16,100	.950
19,580	.800	19,210	.800	16,170	1.000
20,000	.950	19,600	.900		
20,010	1.000	19,750	.950		
20,090	1.100	19,840	1.000		
20,130	1.150	19,870	1.050		
20,150	1.200	19,900	1.100		
20,150	1.250				
Failure at first and third tension members at one end at rivets. One lateral support.		Failure of tension members at one end. One lateral support.		Failure by lateral buckling between supports. Two lateral supports at ⅓ points.	

Test 17B		Test 18A		Test 18B	
Load	Deflection	Load	Deflection	Load	Deflection
250	0.000	250	0.000	250	0.000
1,000	.036	1,000	.036	1,000	.021
2,000	.082	2,000	.082	2,000	.057
3,000	.127	3,000	.123	3,000	.088
4,000	.174	4,000	.167	4,000	.121
5,000	.227	5,000	.222	5,000	.173
6,000	.267	6,000	.258	6,000	.214
7,000	.315	7,000	.295	7,000	.242
8,000	.368	8,000	.337	8,000	.272
9,000	.445	9,000	.384	9,000	.305
10,000	.500	10,000	.437	10,000	.345
11,000	.560	12,000	.555	12,000	.429
12,000	.623	13,300	.650	14,000	.536
13,100	.700	14,400	.750	15,800	.650
14,300	.800	15,100	.850	17,100	.750
15,050	.900	15,250	.900	18,100	.850
15,220	.950	15,320	.950	18,480	.900
15,300	1.000			18,800	.950
				19,050	1.000
Failure by lateral buckling of compression chord. Two lateral supports at ⅓ points.		Failure by lateral buckling of compression chord. One lateral support.		Failure by crushing of compression flange 11″ from side load. One lateral support.	

Logs of tests—Continued

Test 19A-1		Test 19A-2		Test 19B	
Load	Deflection	Load	Deflection	Load	Deflection
312	0.000	250	0.000	250	0.000
1,250	.009	1,000	.010	1,000	.002
2,500	.040	2,000	.045	2,000	.004
3,750	.068	3,000	.078	3,000	.030
5,000	.094	4,000	.107	4,000	.050
6,250	.125	5,000	.144	5,000	.070
7,500	.163	6,000	.170	6,000	.093
8,750	.198	7,000	.205	7,000	.119
10,000	.242	8,000	.248	8,000	.137
11,250	.290	9,000	.303	9,000	.175
12,500	.360	10,150	.350	10,000	.202
13,125	.400	12,400	.450	12,000	.265
13,750	.450	13,800	.550	14,230	.350
14,250	.500	14,430	.650	16,150	.450
		14,580	.750	17,300	.550
		14,520	.800	18,290	.750
		14,300	.900	18,390	.800
				18,440	1.000
				18,350	1.100
Failure by lateral buckling between side load and end reaction. One lateral support.		Failure by lateral buckling. One lateral support.		Failure of compression web members between side and end loads. One lateral support.	

Test 20A		Test 20B		Test 21A	
Load	Deflection	Load	Deflection	Load	Deflection
250	0.000	100	0.000	250	0.000
1,000	.018	1,000	.014	1,000	.015
2,000	.042	2,000	.046	2,000	.040
3,000	.072	3,000	.080	3,000	.074
4,000	.105	4,000	.123	4,000	.108
5,000	.140	5,000	.163	5,000	.140
6,000	.172	6,000	.208	6,000	.170
7,000	.205	7,000	.257	7,000	.200
8,000	.238	8,000	.302	8,000	.231
9,000	.272	9,000	.340	9,000	.264
10,000	.308	10,000	.375	10,000	.292
12,000	.380	12,000	.451	12,000	.355
13,850	.450	13,500	.500	16,400	.500
16,090	.550	14,950	.550	17,450	.550
18,190	.700	16,170	.600	18,860	.650
19,100	.850	18,250	.700	13,340	.400
19,550	1.000	20,000	.800	19,800	.750
19,630	1.050	21,420	.900	20,140	.800
19,700	1.100	22,400	1.000	20,180	.850
19,730	1.150	23,150	1.100		
		23,480	1.200		
		23,510	1.300		
Failure by shear of machine screws in compression flange at fitting. One lateral support.		Failure by lateral buckling. One lateral support.		Failure of rivets in compression flange at center.	

Logs of tests—Continued

Test 21B

Load	Deflection
250	0.000
1,000	.012
2,000	.035
3,000	.060
4,000	.087
5,000	.119
6,000	.150
7,000	.183
8,000	.223
9,000	.265
10,000	.307
12,000	.398
14,290	.500
16,330	.600
17,980	.700
19,160	.800
20,020	.900
20,460	1.000
20,930	1.250
20,980	1.300
20,980	1.300

Failure of rivets in compression flange inside side load.

Test 21C

Load	Deflection
100	0.000
1,000	.013
2,000	.040
3,000	.072
4,000	.100
5,000	.129
6,000	.158
7,000	.185
8,000	.213
9,000	.244
10,000	.268
12,000	.333
14,230	.400
16,650	.500
18,500	.600
19,800	.700
20,420	.800
20,450	.850
20,300	.900

Failure by lateral buckling.
One lateral support.

Test 21D

Load	Deflection
100	0.000
1,000	.013
2,000	.027
3,000	.047
4,000	.070
5,000	.092
6,000	.114
7,000	.134
8,000	.155
9,000	.184
10,000	.213
12,000	.280
14,000	.350
15,150	.400
16,320	.450
18,320	.550
19,800	.650
20,850	.750
21,250	.800
21,800	.900
21,920	.950

Failure by crushing of compression flange.
One lateral support.

Test 22A-1

Load	Deflection
250	0.000
1,000	.023
2,000	.065
3,000	.102
4,000	.137
5,000	.175
6,000	.213
7,000	.254
8,000	.294
9,000	.333
10,000	.376
12,000	.472
13,550	.550
15,300	.650
17,000	.800
17,300	.850
17,500	.900
17,560	----------

Failure of 3d leg from each end in compression.
One lateral support.

Test 22A-2

Load	Deflection
250	0.000
1,000	.027
2,000	.065
3,000	.106
4,000	.148
5,000	.191
6,000	.234
7,000	.275
8,000	.317
9,000	.360
10,000	.404
12,000	.498
14,100	.600
15,050	.650
16,700	.750
17,600	.800
18,180	.850
18,450	.900

Failure by shear of rivets of tension members between end side loads.
One lateral support.

Test 22B

Load	Deflection
250	0.000
1,000	.014
2,000	.044
3,000	.090
4,000	.140
5,000	.175
6,000	.210
7,000	.247
8,000	.283
9,000	.322
10,000	.366
12,000	.466
14,000	.570
15,410	.650
16,710	.750
17,730	.850
18,480	.950
18,790	1.000
19,600	1.650
19,800	----------

One lateral support.
Shear of 2 rivets holding stiffeners to compression flange. Failure of compression flange at point of rivet shear.

Logs of tests—Continued

Test 23A

Load	Deflection
100	0.000
1,000	.028
2,000	.066
3,000	.106
4,000	.142
5,000	.182
6,000	.220
7,000	.257
8,000	.295
9,000	.335
10,000	.380
11,600	.450
13,470	.550
15,000	.650
16,200	.750
17,300	.900
17,550	.950
17,580	1.000
17,000	1.100

Failure by lateral buckling 15″ from center.
One lateral support.

Test 23B-1

Load	Deflection
100	0.000
1,000	.032
2,000	.090
3,000	.129
4,000	.170
5,000	.211
6,000	.256
7,000	.296
8,000	.340
9,000	.385
10,000	.430
12,300	.550
13,750	.650
14,900	.750
15,560	.850
15,600	.900

Failure by lateral buckling.
One lateral support.

Test 23B-2

Load	Deflection
100	0.000
1,000	.037
2,000	.077
3,000	.115
4,000	.155
5,000	.197
6,000	.237
7,000	.277
8,000	.318
9,000	.360
10,000	.405
12,000	.506
13,600	.600
14,800	.700
15,500	.800
15,680	.850
15,760	.900

Failure by lateral buckling.
Two lateral supports at 1/3 points.

Test 30A

Load	Deflection
250	0.000
1,000	.017
2,000	.044
3,000	.073
4,000	.100
5,000	.148
6,000	.182
7,000	.212
8,000	.246
9,000	.283
10,000	.320
12,000	.397
14,200	.600
15,300	.650
16,620	.700
17,830	.750
18,710	.800
19,900	.850
20,640	1.000
20,800	1.050

Failure by crushing of compression chord 8″ from side load.
Two lateral supports at 1/3 points.

Test 30B

Load	Deflection
250	0.000
1,000	.017
2,000	.047
3,000	.075
4,000	.110
5,000	.143
6,000	.173
7,000	.206
8,000	.242
9,000	.277
10,000	.315
12,000	.391
13,380	.450
14,450	.500
15,400	.550
16,800	.650
17,180	.700
17,230	----------

Failure by crushing of compression chord.
Two lateral supports at 1/3 points.

Test 31A

Load	Deflection
312	0.000
1,250	.022
2,500	.052
3,750	.079
5,000	.110
6,250	.145
7,500	.177
8,750	.215
10,000	.260
11,875	.340
12,875	.400
13,440	.440
13,875	.480
14,000	.500
14,125	.600

Failure by lateral buckling.
One lateral support.

Logs of tests—Continued

Test 31B		Test 31C–1		Test 31C–2	
Load	Deflection	Load	Deflection	Load	Deflection
250	0.000	100	0.000	100	0.000
1,000	.023	1,000	.023	1,000	.025
2,000	.055	2,000	.049	2,000	.053
3,000	.090	3,000	.072	3,000	.081
4,000	.131	4,000	.095	4,000	.110
5,000	.165	5,000	.120	5,000	.141
6,000	.197	6,000	.153	6,000	.170
7,000	.230	7,000	.175	7,000	.196
8,000	.260	8,000	.200	8,000	.223
9,000	.293	9,000	.222	9,000	.250
10,000	.328	10,000	.252	10,000	.280
12,000	.412	12,000	.310	12,000	.333
14,000	.523	13,320	.350	14,380	.400
15,600	.650	14,600	.400	15,980	.450
16,450	.800	15,960	.450	17,450	.500
16,700	.900	18,040	.550	19,480	.600
16,500	1.000	18,820	.600	20,300	.650
16,450	1.100	19,490	.650	21,300	.750
16,000	1.200	20,140	.700	21,670	.800
		20,480	.750	21,940	.850
				22,120	.900
				21,900	1.000
Failure by lateral buckling. Two lateral supports at ⅓ points.		Failure of end fittings. One lateral support.		Failure by bending of a section of the compression chord. One lateral support.	

Test 32A		Test 32B		Test 40A	
Load	Deflection	Load	Deflection	Load	Deflection
250	0.000	250	0.000	250	0.000
1,000	.020	1,000	.025	1,000	.027
2,000	.052	2,000	.065	2,000	.070
3,000	.086	3,000	.095	3,000	.120
4,000	.120	4,000	.132	4,000	.170
5,000	.155	5,000	.170	5,000	.217
6,000	.195	6,000	*.204	6,000	.265
7,000	.237	7,000	.278	7,000	.314
8,000	.280	8,000	.320	8,000	.357
9,000	.325	9,000	.364	9,000	.410
10,000	.365	10,000	.452	10,000	.472
12,000	.405	12,000	.548	12,000	.618
14,000	.515	14,000	.650	13,750	.800
16,080	.700	15,680	.700	14,800	1.000
17,100	.900	16,350	.800	15,050	1.100
17,240	.950	17,620	.900	15,200	1.200
17,250	1.000	18,670	1.000	15,270	1.250
17,180	1.050	19,430	1.050	15,240	1.300
17,090	1.150	19,620	1.100		
16,990	1.250	19,660			
Failure by crushing of the compression chord. One lateral support. Two lateral supports at ⅓ points.		Failure by crushing of compression chord at fitting. Two lateral supports at ⅓ points. *0.240.		Failure by bending of a section of the compression chord at center. Two lateral supports at ⅓ points.	

Logs of tests—Continued

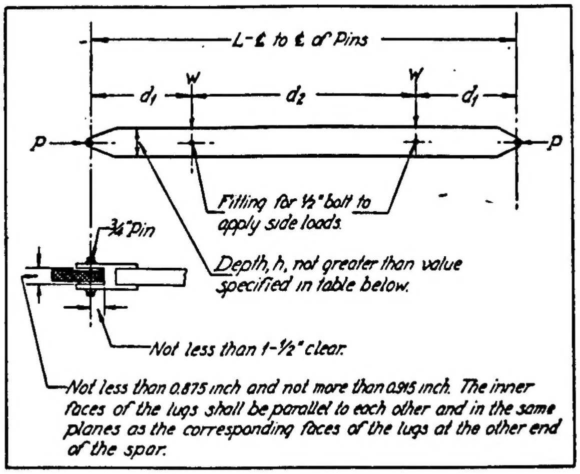
Test 40B			
Load	Deflection	Load	Deflection
100	0.000	13,720	0.700
1,000	.025	15,100	.850
2,000	.064	16,080	1.000
3,000	.105	17,080	1.200
4,000	.150	18,040	1.600
5,000	.193	18,280	1.750
6,000	.232	18,300	1.800
7,000	.270	18,280	1.850
8,000	.316	19,150	1.950
9,000	.380	20,350	2.050
10,000	.445	21,000	2.150
12,000	.590	22,350	2.300

Failure by excessive deflection.
Side load block hit upright support.
1 lateral support.

APPENDIX II

REQUIREMENTS FOR EXPERIMENTAL METAL SPARS FOR PURSUIT AND OBSERVATION LOADINGS

SECTION I. *Dimensions and Loading—General requirements.*—The required dimensions and loads are given in the figure and table below for both the pursuit and observation loadings.



Loading	Loads		Dimensions			
	P	W	L	h	d_1	d_2
			Inches.	*Inches.*	*Inches.*	*Inches.*
Pursuit	7,000	700	84	4	20	44
Observation	20,000	2,000	96	6.25	22.5	51

In designing the spar the end and side load pins are to be located at the neutral axis of the section, and no initial camber is to be given the spar.

The spar will be braced laterally at midspan only; that is, laterally unsupported length equals L/2 inches.

The spar shall be so designed as to sustain a 50 per cent reversal of the side load, no axial load being present in this condition.

In obtaining the weight of the spar for computing the strength-weight ratio, 6 inches will be cut from each end, making the basic length L–12 inches. The fittings for the side load attachments, exclusive of the ½-inch bolts, will be included in the spar weights.

SEC. II. *Requirements as to tests on minor parts and properties of sections.*—In order to make a comparison of the various types of spars tested, from the standpoint of their computed and allowable strengths, it is necessary to have data as to the strength properties of the materials entering into the construction of the spar. These data must be obtained from tests on minor parts taken from the spar itself or from sections taken from the identical material from which the various parts of the spar are made. Because of the variety of shapes and sections used, it is impossible to specify any set of tests to cover all types of spar, but the following list gives an outline of the test data desired with each test spar. Where the test requirements will obviously not apply, or where other data appear more desirable, the contractor shall consult the division as to the exact interpretation of the requirements to be applied to his spar.

DATA REQUIRED

1. Modulus of elasticity in tension of the material in the compression chord, the tension chord, and the web of the spar.

2. Proportional limit or yield point, if possible both, of the material in the compression chord, the tension chord, and the web as obtained from tests in tension and compression, the compression test to be made on a member having flat ends and a length equal to at least four times its least diameter.

3. Ultimate strength in compression of a section of the spar with flat ends, and a length equal to twice the depth of the spar, unless some feature of the design makes it desirable to increase or decrease this length. When it is impossible to make the compression tests called for in paragraph 3 on the chord or web members as, for instance, in a box section where all elementary members are flat sheets, this test on a section of the built-up spar may be substituted for those compression tests. The specimen subjected to this test shall be furnished to the division with the spar on order and the other accompanying data. The following data will also be required as to the properties of the section and the computations substantiating them must be submitted to the division. These data shall be checked against the actual measurements of the spars furnished.

1. Moment of inertia of the whole section about its major and minor axes and the moments of inertia of each constituent part taken about the axes passing through the centroid of each such part and parallel to the major and minor axes of the whole section.

2. Location of the major and minor axes of all unsymmetrical or complex sections incorporated in the spar.

3. Areas of the whole section and of each constituent part.

Sec. III. *Sequence of designing and testing.*—In submitting bids for a spar of a new type of construction, the designer need furnish only a sketch showing the essential features of the type of construction proposed.

In submitting bids for a modification of a type of construction previously tested the designer shall furnish a dimensioned drawing showing the cross sections of all the main members of the spar and computations to show that the revised design will show a higher strength-weight ratio or a greater degree of stiffness than the similar spars previously tested.

Whenever two spars of the same type are ordered at the same time the construction of the second shall not be commenced until the designer has received a report of the results of the test of the first.

Care must be taken by the designer to provide end fittings that will not fail in the tests. If any failures or permanent deformations, such as elongated bolt holes, are noticed in the 6-inch portions that are cut off before weighing, the spar will be returned to the designer for rebuilding at his expense. (Several types of spar have been furnished with weak end fittings that failed and prevented the real merits of the spars in question from being determined by the tests. This was absolutely unnecessary, as the use of extra heavy construction in the end fittings is not reflected in the weight recorded for the spar. In fact, it was in order to prevent having such failures and to encourage the use of extra strong end fittings that 6 inches are cut off of each end before weighing.)

Sec. IV. *Preliminary bending tests.*—When the purchase order or contract calls for preliminary bending tests the contractor shall make bending tests to destruction on two sections of the spar, one of such length that failure will occur in a chord member and the other of such length that failure will occur in the web.

These tests shall be made on sections of spar of the same material and cross section as the spar to be submitted to the division, and in the following manner: The sections shall be tested as simple beams with two symmetrically located bending loads, each 0.1 L from the center of the span, where L is the span. The bending loads shall be applied through fittings identical to the side load fittings used in the spar submitted to the division. The end fittings shall be sufficiently rigid to prevent their permanent deformation in the tests. The spars shall be given sufficient lateral support to prevent failure by lateral buckling of the compression chord. The specimens used for these tests and complete test logs shall be furnished to the division.

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